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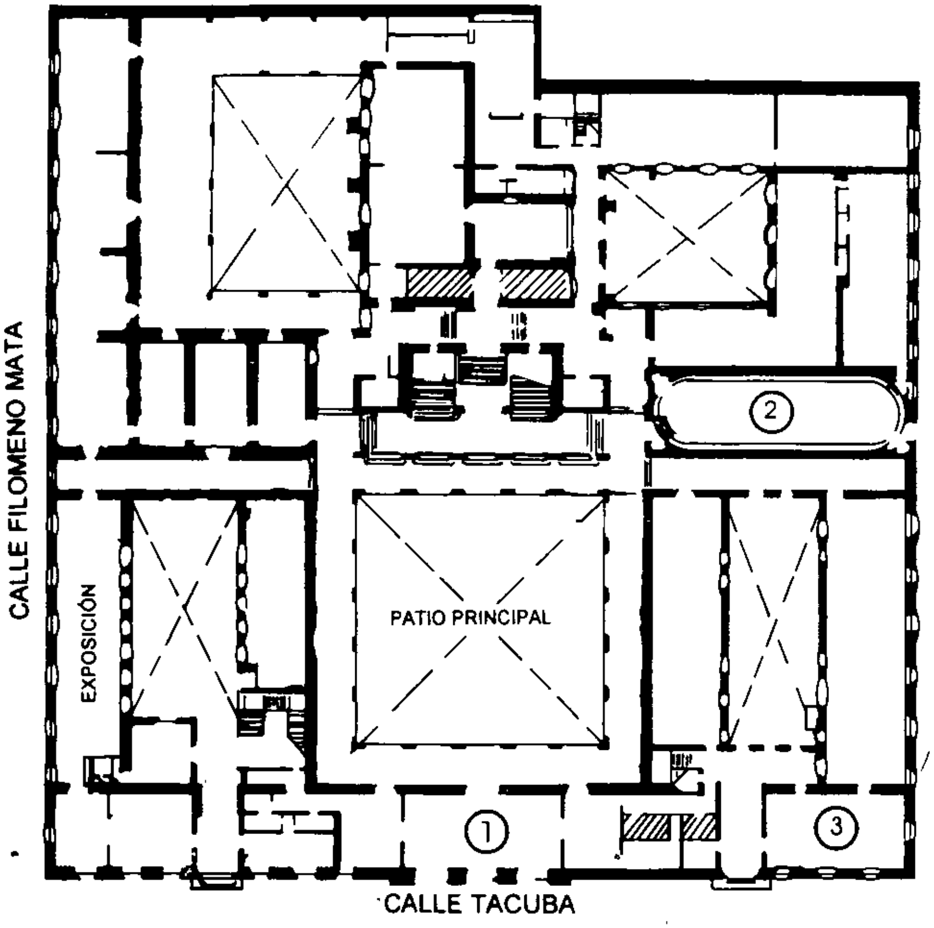

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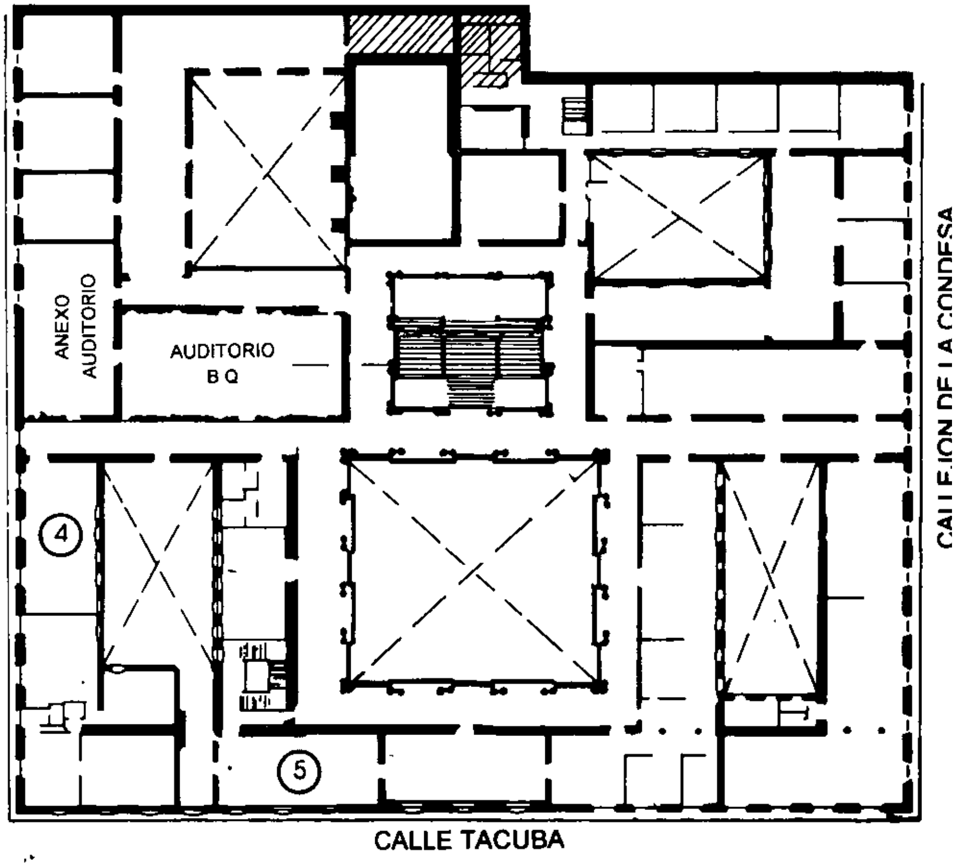

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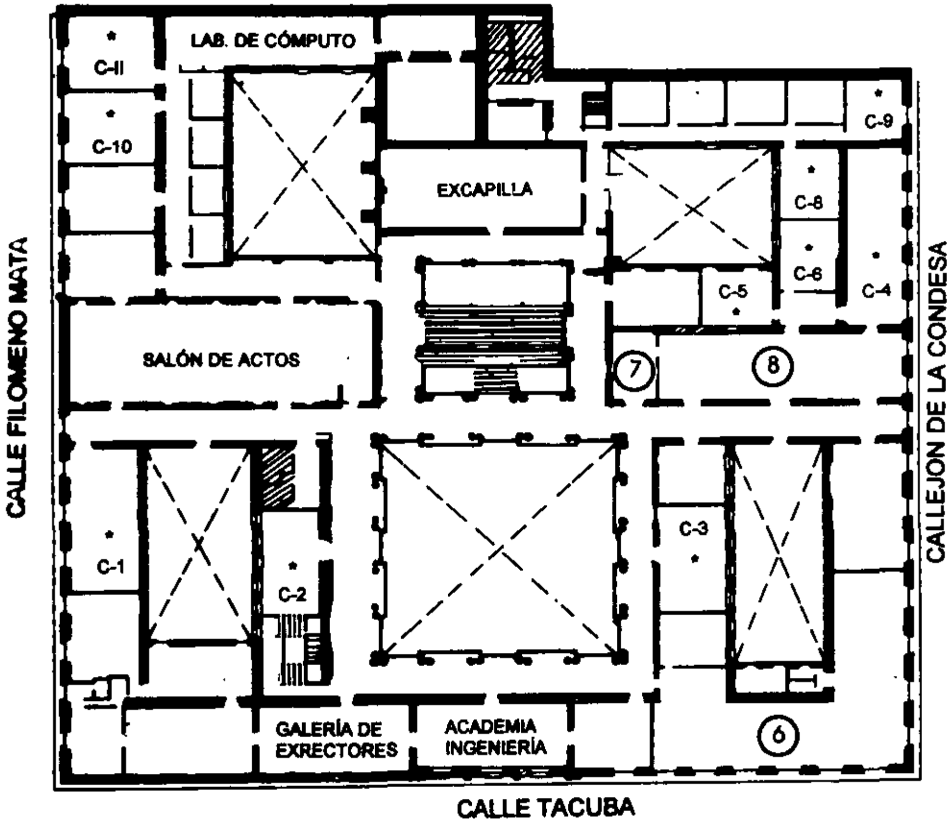

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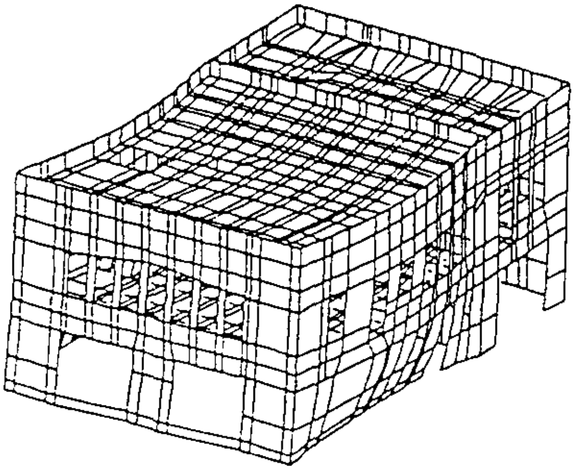

Figure. 10. Finite element model.

stresses in masonry given by American codes. It is reasonable to assume with these stress levels that little or no cracking should have occurred.

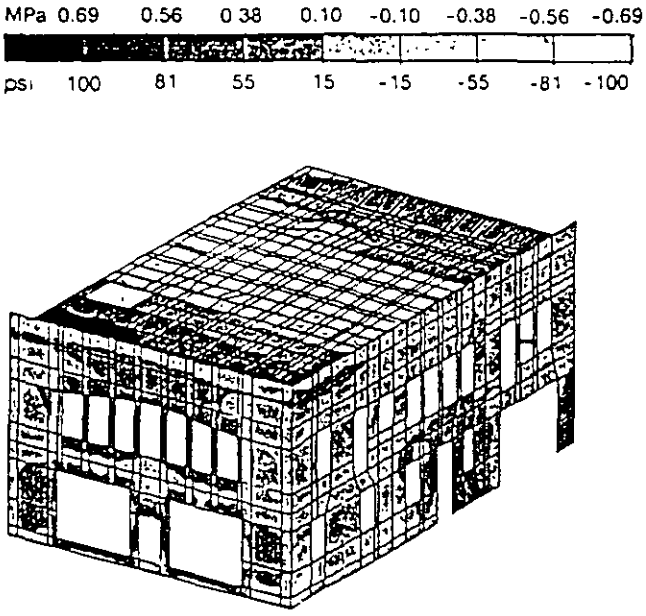

Figure 11. Shear stress contours.

Stress contours are not shown for the cases where the FEM model was run with equivalent static forces, or with a response spectra analysis, however, good agreement was seen between them and those for the time-step integration analysis. This suggests that the discrete MDOF model may be suitable for estimating peak lateral forces despite its simplistic assumptions. Moreover, the linear response spectra approach proved admissible once modes of vibration could be identified.

6 CONCLUSIONS

Various analytical techniques used to study response and damage of the historic unreinforced masonry structure helped to explain why the building withstood the Loma Prieta Earthquake with little damage. Wall shear stress levels were shown by both crude and sophisticated methods to be within reconcilable ranges.

The recorded response and the overall behavior of the structure during the earthquake were estimated well with the discrete MDOF linear dynamic analyses. Though it was far simpler than the finite element model, salient characteristics of response such as diaphragm flexibility, soil-structure interaction and lateral stiffness of walls with openings could be represented.

The study has inferred that the vulnerability of unreinforced masonry construction east of the Rockies may not be as severe as believed with the common consensus. There is promise that future earthquake hazard studies for specific buildings may show that demolition or expensive rehabilitation may not be necessary.

7 ACKNOWLEDGEMENTS

The research was funded by the National Science Foundation of the United States under grant #BCS 90-03654. Appreciation is extended to the California Strong Motion Instrumentation Program for providing the acceleration records, and to Mr. Kurt Auslinger, the owner of the building, for providing information on the structure.

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XXVI CURSO INTERNACIONAL DE INGENIERIA SISMICA

MÓDULO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

ESTIMATING RESPONCE OF MASONRY STRUCTURES WITH LINEAR FINITE ELEMENTS

EXPOSITOR: DR. ARTURO TENA COLUNGA
PALACIO DE MINERIA
SEPTIEMBRE DE 2000

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FIFTH NORTH AMERICAN
MASONRY CONFERENCE
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June 3-6, 1990

ESTIMATING RESPONSE OF MASONRY STRUCTURES WITH LINEAR FINITE ELEMENTS

Arturo Tena-Colunga[1] *and Daniel P. Abrams*[2]

1. ABSTRACT

Dynamic response of two, three-story reinforced masonry structures that were subjected to simulated earthquake motions was correlated with results of numerical models. Three dimensional elastic finite elements were used to model perforated flanged walls. Computed frequencies and mode shapes are compared with those obtained from experiments. The influence of different lateral force distributions on normal and shear stresses at the piers was also examined with the finite element models. In addition, lateral drifts were computed using the FEM which were correlated with measurements. Research was part of Task 7.1 of the U.S.-Japan coordinated masonry research program.

2. INTRODUCTION

A number of analytical models for perforated shear walls (commonly known as a "pier models") do not replicate the rotational restraint at the top and bottom of piers. Frame type models for perforated walls also overestimate lateral stiffness because intersections of pseudo beam and column members must be considered rigid. To avoid these artificial restraints, a perforated wall can be represented with

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plane stress finite elements. If an in-plane wall contains a flange, shell elements need to be used to model both bending and membrane actions of the flange.

It may not be possible to accurately model behavior of reinforced masonry structures with linear elastic models because of effects related to cracking, sliding along bed joints and slippage of reinforcement. The purpose of this paper is to help illustrate the utility of linear elastic finite shell elements for estimating response and behavior of low-rise reinforced masonry building structures. An attempt is made to show the merits and limitations of using a linear elastic, and thus approximate, finite element model to represent the more complex behavior and response of reinforced masonry within both linear and nonlinear ranges. Computed frequencies and mode shapes are compared with those obtained from experiments. The influence of different lateral force distributions on normal and shear stress distributions acroos a story is examined with the finite element model. Computed drifts for an inverted triangular load distribution are compared with those measured on the shaking table.

As a part of Task 7.1 of the TCCMaR program, one-quarter scale, three-story, reinforced masonry structures were subjected to simulated seismic loading. A full description of the experimental program and its preliminary results are described in references 1, 2, 3 and 5 . As is shown in Fig. 1, test structure RM1 was a fully symmetric structure that consisted of flanged walls with window openings, while test structure RM3 had an asymmetric layout with window and door openings. The models were designed based on flexural criteria, in order to show that reinforced masonry could be designed to incur inelastic deformations. Simulated earthquakes were based upon the one recorded at El Centro, California (NS component), during the 1940 Imperial Valley Earthquake. The amplitude and time scale of the record was adjusted so that the structures would respond in a desired range.

3. DESCRIPTION OF FINITE ELEMENT MODEL

Four-noded (6 dof/node), rectangular flat-shell, finite elements were used to model the inplane perforated walls, flanges and slabs. This element combined both flexural and membrane actions (a complete description can be found in Ref. 4). Symmetry with respect to the lateral force distribution was taken into consideration, and therefore, just half of the structure was modeled. The mesh for structure RM consisted of 138 elements and 180 nodes, whereas the mesh for structure RM3 consisted of 108 elements and 150 nodes(Fig. 2). The meshes were coarse for representing stress values quantitatively, however, they were felt to be adequate for defining qualitative stress patterns and for estimating deflections. A consistent mass formulation was used in the analysis for distributing the self weight of the elements, and the additional weight supported by the slabs. The modulus of elasticity used in the analysis was 721 ksi, which was obtained from the average of prism tests(Ref. 1). A Poisson ratio of 0.28 was considered.

4. FREQUENCIES AND MODE SHAPES

Frequencies for the first three translational modes were computed and are in Table 1. The apparent first mode frequencies measured during the free vibration tests before all earthquake test runs of structures were found to be 15 Hz and 13.5 Hz for models RM1 and RM3 respectively. The analytical models were always stiffer than the real structures. These differences may be attributed to premature cracking of the models due to shrinkage. In addition, the flexibility of the shaking table platform and support system was neglected in the analysis. Also, finite elements are always stiffer than the exact solutions.

TABLE 1.- COMPUTED FREQUENCIES FOR FIRST THREE TRANSLATIONAL MODES (Hz)

MODE	MODEL RM1	MODEL RM3
1	19.6	15.6
2	58.0	48.6
3	96.4	80.9

The fundamental modes shapes obtained from the finite element analysis are presented in Fig. 3..They are compared to normalized shapes measured during shaking in Fig. 4. The modal participation factor $\alpha_{i,}$ for any mode shape i, is defined as $\alpha_i = \Sigma m_j \Phi_{ij} / \Sigma m_j \Phi_{ij}^2$, where Φ_{ij} is the amplitude of mode shape i at level j, and m_j is the mass at level j.

Computed modal participation factors for normalized first mode shapes are compared in Table 2 for measured and calculated values. Correlations for both structures suggested that the modal participation factor for the first mode was insensitive to the shape assumption, and thus, a coarse mesh model was felt to be appropriate.

TABLE 2.- MODAL PARTICIPATION FACTORS FOR THE FIRST MODE SHAPE

Model	Measured	Calculated	% Difference
RM1	1.30	1.28	1.3%
RM3	1.28	1.28	0.5%

5. VERTICAL AND SHEAR STRESSES

From lateral acceleration measurements at each of the three levels, it was apparent that force distributions fluctuated substantially during shaking. Therefore, it was of interest to study the sensitivity of shear and normal stress distributions at the base story to variations in the lateral force distribution. The question was " does the lateral distribution influence damage at the base which, in turn, influences the lateral force distribution ? ".

The same finite element meshes used for the frequency analysis were used to study the influence of different lateral force distributions on normal and shear stresses at the base story, where most of the damage was observed. Lateral forces were distributed across the nodes of each floor slab. Three hypothetical lateral force distributions were studied: (a) inverted triangular, (b) uniform, and (c) triangular. The analyses were done using forces that would result in the same maximum base shear as measured during the last test run of the shaking table tests. The computed stress contours are shown in Figs. 5 and 6. A summary of the results obtained are presented in Table 3.

The vertical normal stresses can be analyzed from Figs. 5 and 6 for the different load distributions under study. For all distributions on both models, the tension and compression stress concentrations were located nearby the corners of an opening, as well as in the vicinity of the corner of the exterior piers at the base level, as was expected. The inverted triangular load distribution produced higher stress concentrations at the first story than the triangular or uniform load distribution.

TABLE 3.- MAXIMUM STRESSES FOR DIFFERENT CONDITIONS AND DISTRIBUTIONS OF LATERAL LOADING(FROM FINITE ELEMENT ANALYSIS)_

LATERAL LOAD DISTRIBUTION	a) EQUIVALENT BASE SHEAR MODEL RM1 (V_1 = 1.65 W)				MODEL RM3 (V_1 = 1.47W)			
	$\tau_{XY}(+)$	$\tau_{XY}(-)$	$\sigma_Y(+)$	$\sigma_Y(-)$	$\tau_{XY}(+)$	$\tau_{XY}(-)$	$\sigma_Y(+)$	$\sigma_Y(-)$
Inverted Triangular	210 (1.45)	66 (0.46)	375 (2.59)	375 (2.59)	306 (2.11)	70 (0.48)	759 (5.23)	932 (6.43)
Uniform	210 (1.45)	59 (0.41)	332 (2.29)	332 (2.29)	299 (2.06)	68 (0.47)	678 (4.67)	880 (6.07)
Triangular	208 (1.43)	51 (0.35)	247 (1.70)	247 (1.70)	287 (1.98)	65 (0.45)	548 (3.78)	778 (5.36)

NOTES :
Stress units : psi (MPa)
(+) Tension

The magnitude of the compressive vertical stresses at maximum base shear condition for the inverted triangular force distribution (Table 3) may be considered as moderate for structure RM1 (375 psi), and important for model RM3 (932 psi).

The analyses indicated that structures RM1 should have cracked at a base shear equal to 0.43W and that structure RM3 should have cracked at a base shear of 0.2W (assuming a tensile strength equal to 100 psi). These values of base shear were lower than those for the initial test runs when no cracking was observed (0.41W and 0.38W for RM1 and RM3 respectively).

Computed shear stresses are presented in Figs. 5 and 6 for the three different load distributions and an equivalent base shear. The shear-stress contours identify the stress concentration regions in the vicinity of openings, particularly in the central pier at the base. The stress contour at the first story are approximately the same for all three lateral distributions. The central pier of RM1 should be subjected to a high shear stress concentration in the zone located between adjacent windows. In the first story, the average shear stress in that zone should be almost twice as much as the average shear stress for the exterior piers. The magnitude of the maximum shear stresses (Table 3) corresponded to the peak shear stresses obtained from diagonal compression tests of square reinforced panels (Ref. 1). However, the damage observed did not correlate well with the stress concentration regions defined by the finite element analysis. In RM1, exterior piers failed in diagonal tension while the central pier could not attract the shear that was expected due to the sliding along the bed joints at the top and bottom levels of window openings (Fig. 7). This sliding mechanism was not modeled with the elastic finite elements.

6. LATERAL DEFLECTIONS

The maximum deflection at the top of each structure was computed using the linear elastic finite element model with an inverted triangular load distribution. The magnitude of the loads was determined according to the base shear measured experimentally for each run. For the first test run, uncracked elements were used. In subsequent runs, cracking was considered by using a simplified approach. The moment of inertia of the cracked section, I_{cr}, was considered instead of the moment of inertia of the gross section, I_g, for those piers which were observed to be cracked. An equivalent modulus of elasticity ,E_{cr}, was input to the computational model ;

$$E_{cr} = (I_{cr} / I_g) E_m$$

It should be noted that the axial stiffness of elements subjected to tension was overestimated, while the axial stiffness of those subjected to compression was underestimated. However, these effects should balance by themselves because of the rather symmetric mapping considered.

Measured and computed maximum drifts for models RM1 and RM3 for each test run are presented in Table 4. Computed drifts are always smaller than measured values. The correlation between measured and computed drifts for small amplitude test runs was good. Differences between measured and computed drifts at failure were evident and show the limitation of using cracked-elastic models together with a static analysis in the forecasting of drift for inelastic structures subjected to dynamic excitation. The correlation between measured and computed drifts is better for structure RM1 than for structure RM3. This seems reasonable since RM1 did not yield until run 3, while RM3 should have yielded during run 2.

TABLE 4. - MEASURED VS COMPUTED MAXIMUM DRIFTS (δ_3 / H)

MODEL RM1

Run	$\frac{V_b}{W}$	$\frac{M_b}{M_y}$	Measured Drift (%)	FEM Drift (%)
1	0.41	0.37	0.03	0.02
2	0.72	0.73	0.09	0.08
3	0.97	0.81	0.19	0.15
4	1.65	1.15	1.06	0.42

MODEL RM3

Run	$\frac{V_b}{W}$	$\frac{M_b}{M_y}$	Measured Drift (%)	FEM Drift (%)
1	0.38	0.55	0.04	0.03
2	0.72	1.01	0.17	0.10
3	0.92	1.29	0.41	0.19
5	0.99	1.34	0.49	0.22
6	1.47	1.58	1.32	0.48

7. SUMMARY AND CONCLUSIONS

The study showed that linear elastic finite elements :

a) Could give reasonable estimates of natural frequency.

b) Could give excellent estimates of mode shape and modal participation factor.

c) Could not give good predictions of damage patterns, because of inelastic sliding and yielding mechanisms .

d) Could give good estimates of lateral drift for low-amplitude excitations, but not for large-amplitude ones.

Future research on analytical modeling of masonry should include development of inelastic elements that can represent cracking, yielding of reinforcement, sliding mechanisms and diagonal tension.

8. ACKNOWLEDGEMENTS

The research project was part of Task 7.1 of the US-Japan Coordinated Masonry Research Program. The project was funded by the National Science Foundation under grant ECE-86-12476. Opinions and findings presented in the paper are those

of the authors, and may not necessarily reflect consensus opinion of the Technical Coordination Committee for Masonry Research.

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4.- Lopez, L. , R. Dodds, D. Rehak and R. Schmidt ,*"POLO-FINITE, A Structural Mechanics System for Linear and Nonlinear, Static and Dynamic Analysis"*, Civil Engineering Systems Laboratory, University of Illinois at Urbana-Champaign.1987.

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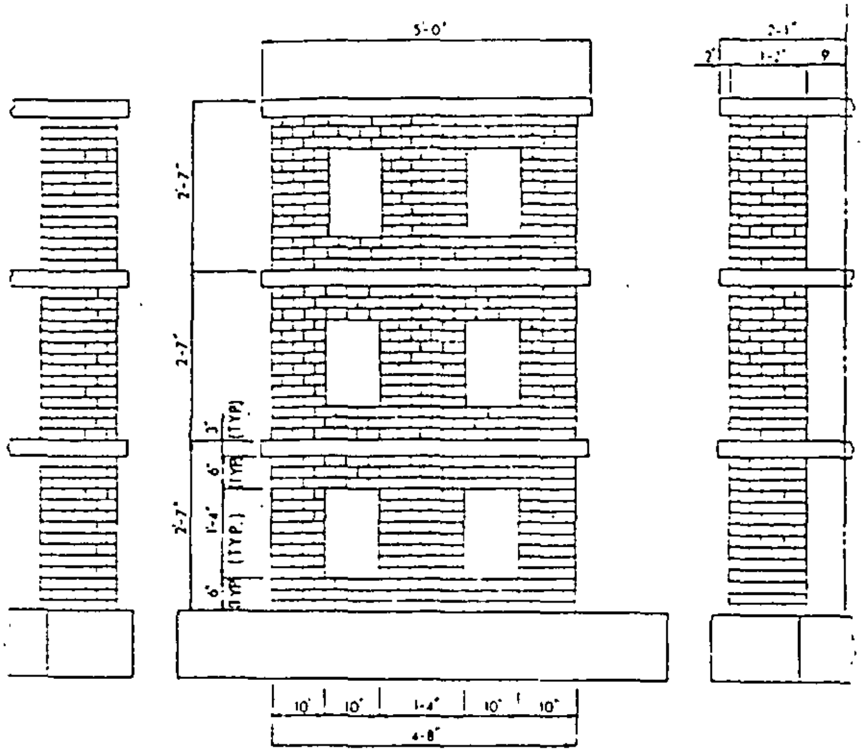
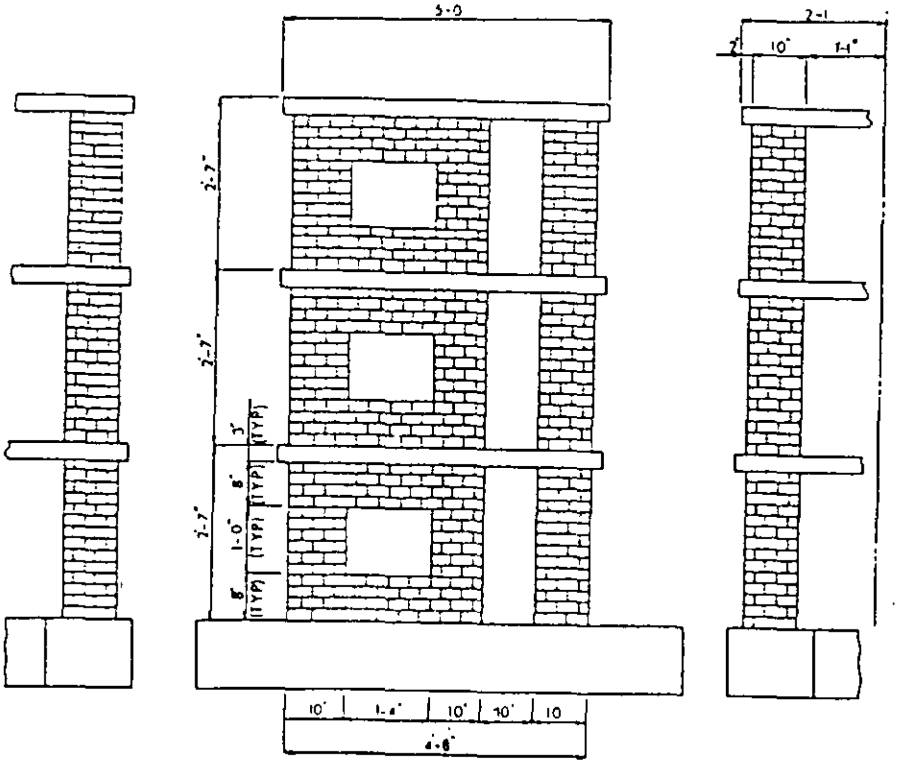
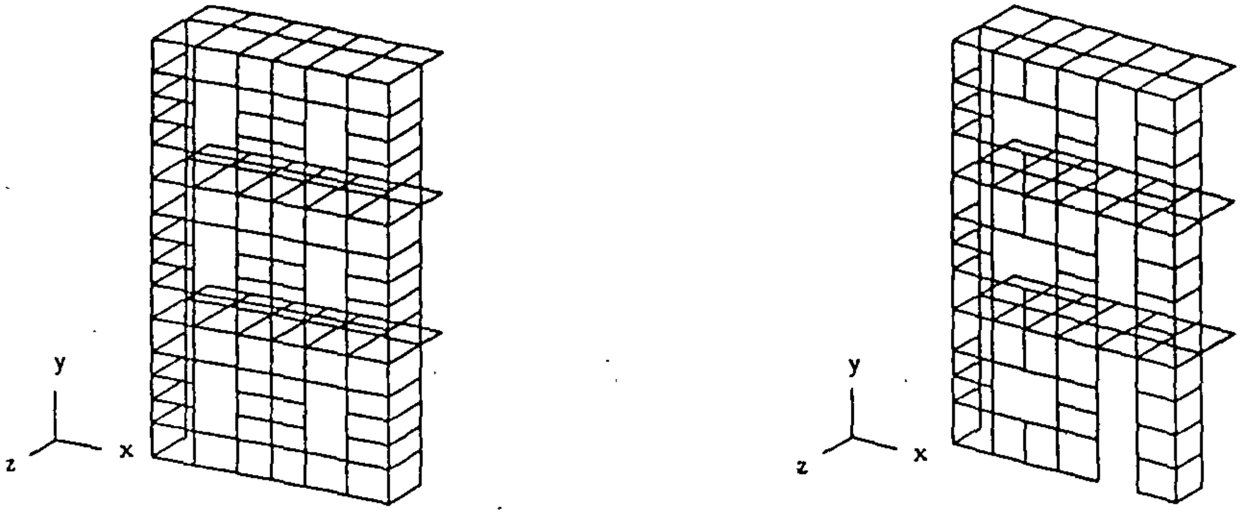


a) Structure RM1



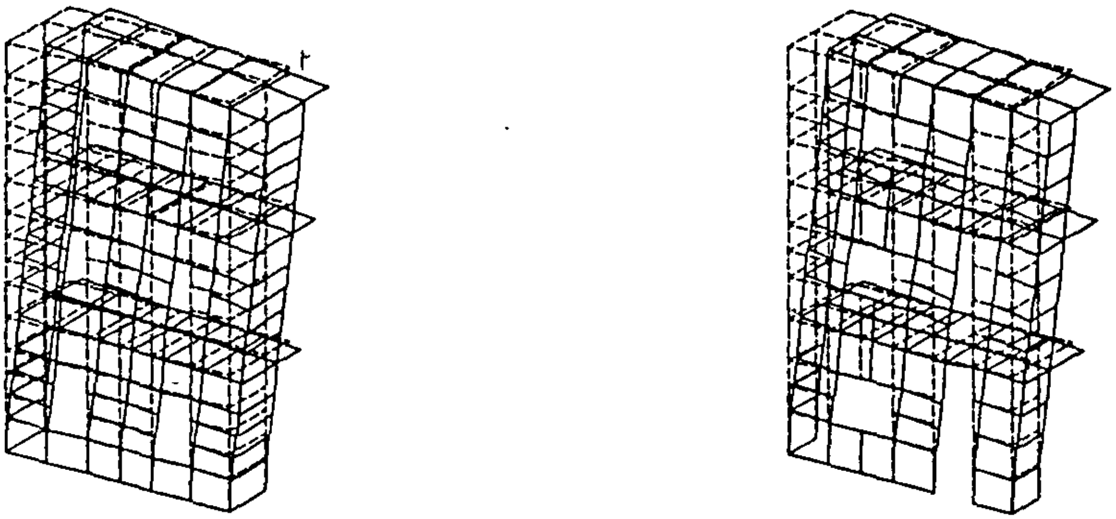
b) Structure RM3

Fig. 1.- Vertical Layout of the Test Structures



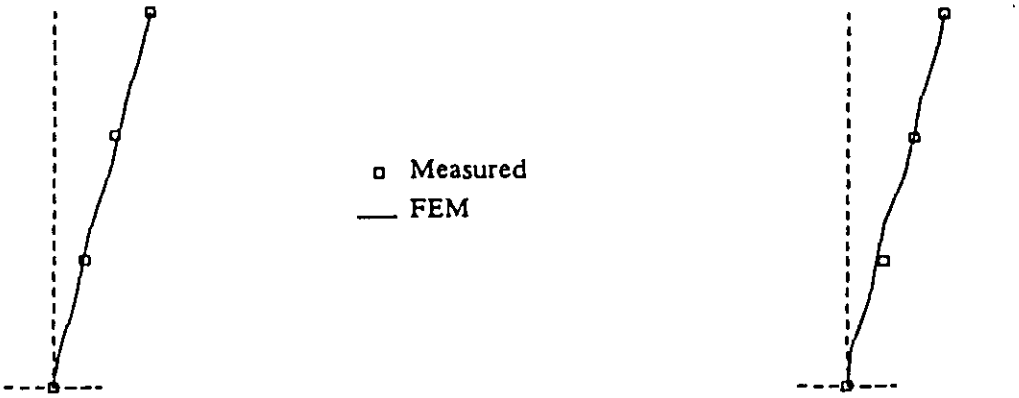
a) Model RM1 b) Model RM3

Fig 2.- Finite Element Meshes



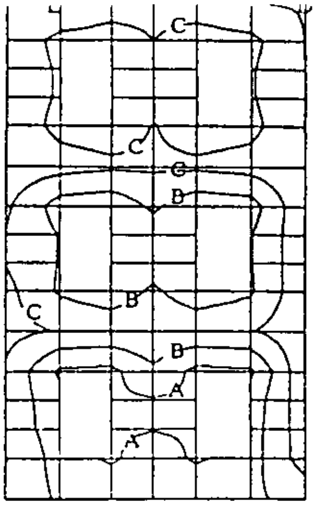
a) Model RM1 b) Model RM3

Fig 3.- First Mode Shape obtained from FEM



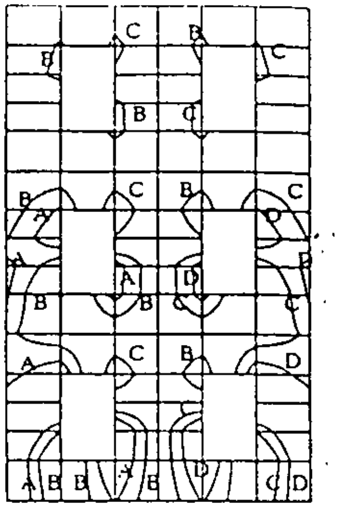
a) Model RM1 b) Model RM3

Fig 4.- Normalized First Mode Shape, measured vs FEM

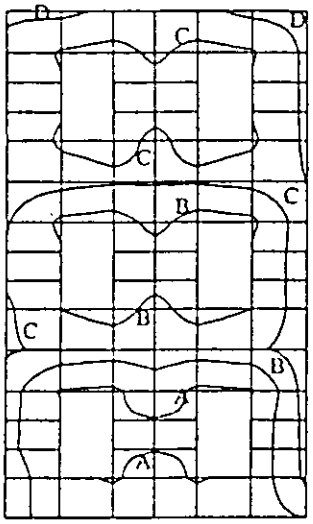


Contour values (psi) :

A = 194	A = 194
B = 136	B = 97
C = 78	C = -97
D = 20	D = -194



a) Inverted Triangular



Contour values (psi) :

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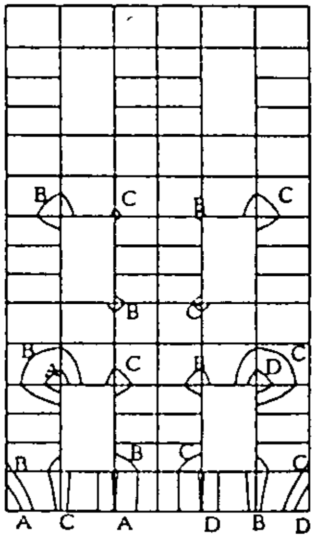
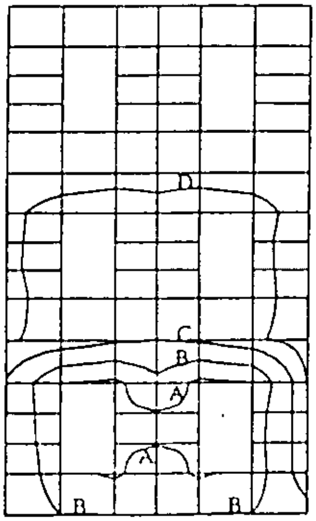


b) Uniform



Contour values (psi) :

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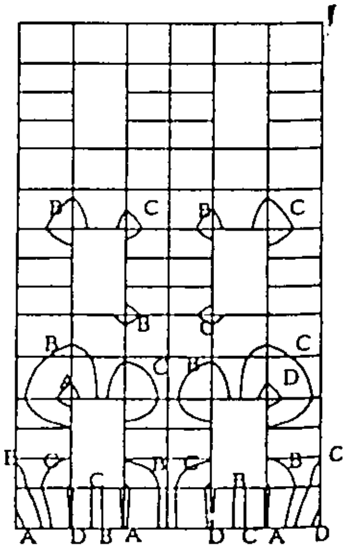
C = 78	C = -39
D = 20	D = -175

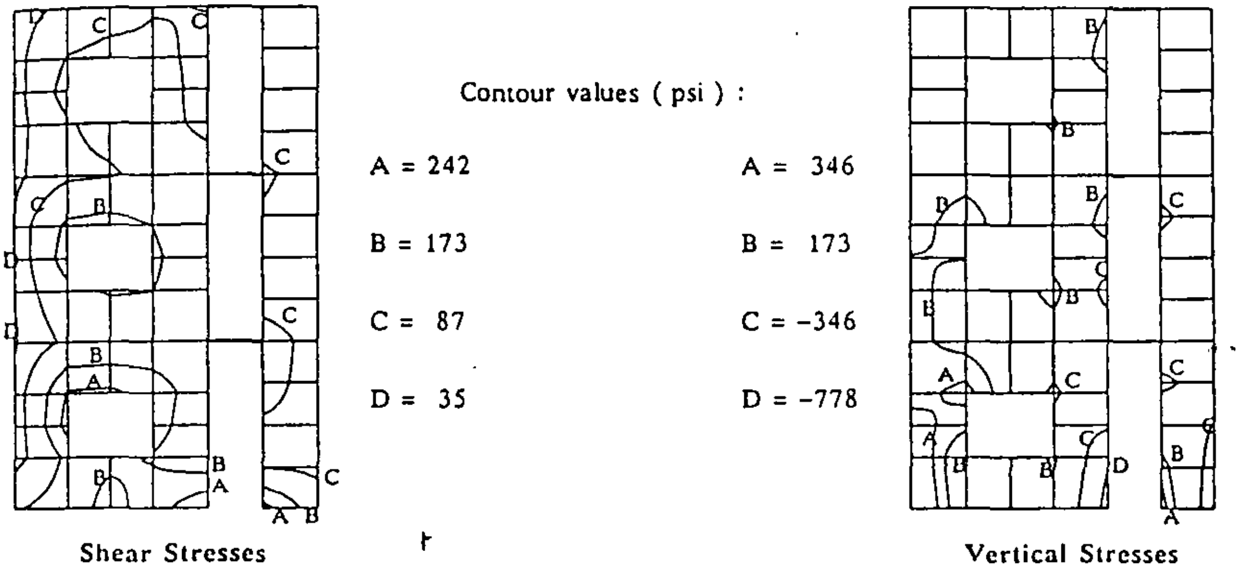


c) Triangular

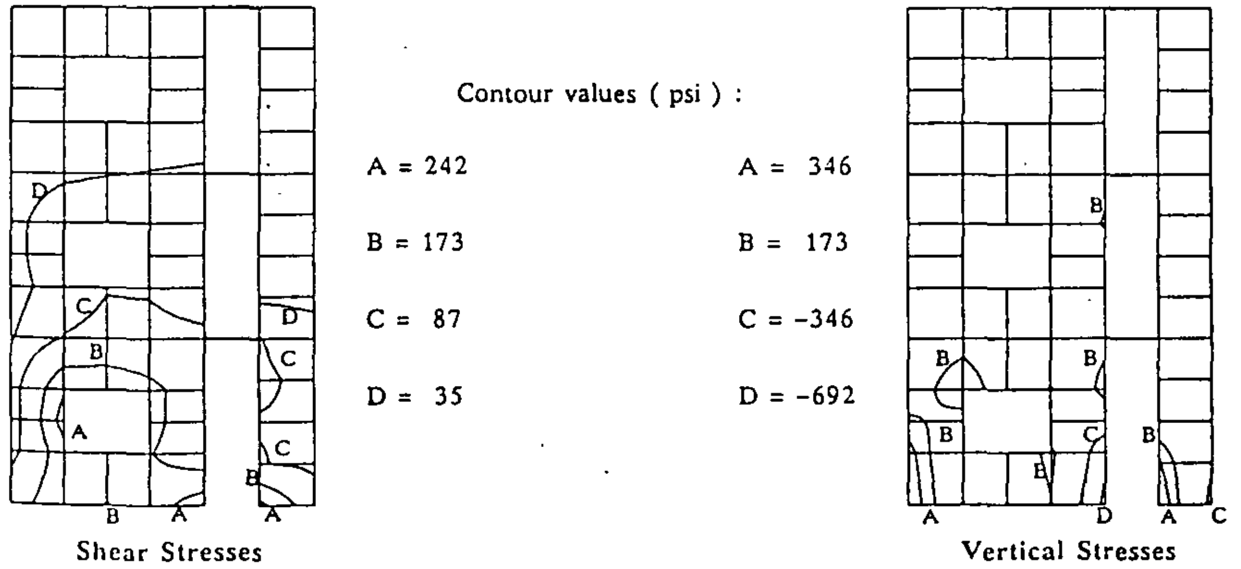
Shear Stresses

Vertical Stresses

Figs. 5.- Stress Contours for different lateral force distributions, Structure RM1

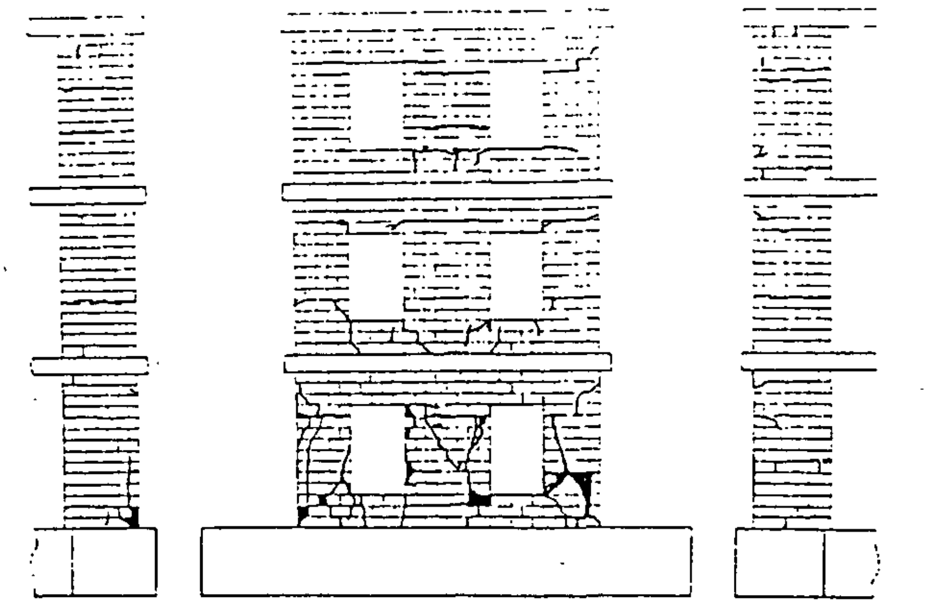


a) Inverted Triangular

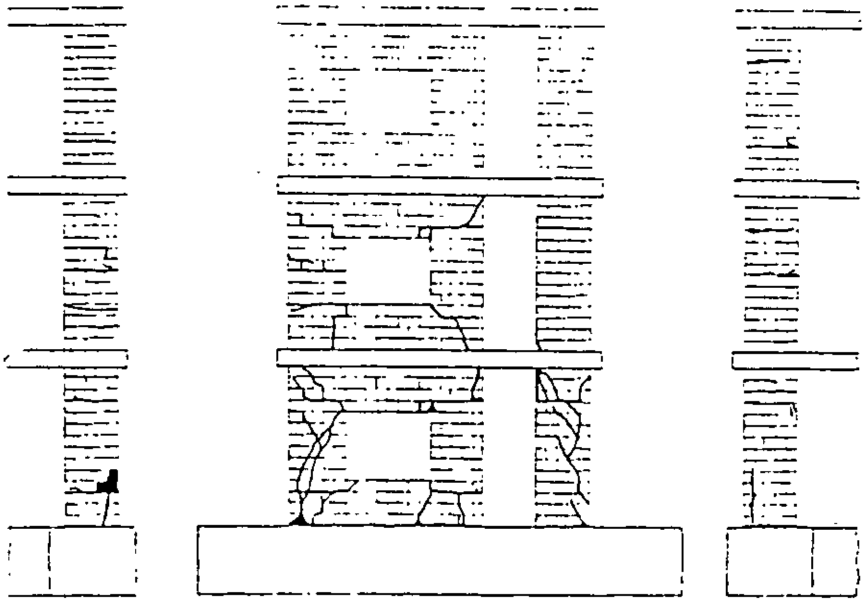


b) Triangular

Figs. 6.- Stress Contours for different lateral load distributions, Structure RM3



a) Structure RM1



b) Structure RM3

Fig. 7. Observed damage of reduced scaled structures during shaking table tests

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XXVI CURSO INTERNACIONAL DE INGENIERIA SISMICA

MÓDULO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

STRUCTURAL EVALUATION OF LOW RISE MASONRY BUILDINGS WITH FLEXIBLE DIAPHRAGMS SUBJECTED TO EARTHQUAKES

EXPOSITOR: DR. ARTURO TENA COLUNGA
PALACIO DE MINERIA
SEPTIEMBRE DE 2000

PROCEEDINGS

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Calgary, Alberta, Canada

5-7 July, 1994

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STRUCTURAL EVALUATION OF LOW-RISE MASONRY BUILDINGS WITH FLEXIBLE DIAPHRAGMS SUBJECTED TO EARTHQUAKES

Arturo Tena-Colunga[1] and Daniel P. Abrams[2]

1. ABSTRACT

Paper presents a comprehensive methodology for the structural evaluation of low-rise masonry structures with flexible diaphragms when subjected to moderate or strong earthquake motions. A discrete multi-degree-of-freedom dynamic model was developed to reproduce the dynamic response of low-rise masonry buildings with flexible diaphragms The proposed discrete model has been used both to compute time-history responses of any structural element represented on it as well as to determine peak responses, or responses at specific times of interest Based on the dynamic responses computed with the discrete models, different types of evaluation of particular structural components, or the structure as a whole, can be performed In-plane and out-of-plane stress evaluations of masonry components can be done using the results computed from the discrete models together with a suitable stress model. The case studies of a low-rise masonry buildings with flexible diaphragms which obtained instrumented records from the 1989 Loma Prieta Earthquake are presented to help illustrate the proposed methodology

2. INTRODUCTION

It is common practice among structural engineers to target masonry construction as vulnerable and unreliable when subjected to moderate or strong earthquakes, particularly unreinforced masonry (URM) structures This belief seems to be based on the extent of damage experienced by some URM buildings during major earthquakes in the past. However, masonry structures can behave well when subjected to strong ground motions, including the unreinforced type, as it has been witnessed in recent strong earthquakes such as the 1985 Michoacán Earthquake in México (confined masonry construction) and the 1989 Loma Prieta Earthquake in California (some low-rise URM buildings).

One of the reasons that structural engineers may have to condemn old masonry structures without taking any technical consideration is the lack of confidence on using standard analytical methods to evaluate stresses in masonry buildings subjected to lateral

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loading. The ABK Methodology (ABK, 1981) and the UCBC Code (SEAOC, 1990) have provided the first set of recommendations to evaluate old masonry construction, however, their influence is very regional and their procedures might seem crude for some and conservative to others. Dynamic effects are virtually no considered in these provisions.

This work presents a methodology for the structural evaluation of low-rise masonry structures with flexible diaphragms when subjected to moderate or strong earthquake motions. The methodology intends to be comprehensive, considers the dynamic response of masonry structures, and is based upon MDOF discrete dynamic models as outlined in detail elsewhere (Tena-Colunga, 1992 a/b, Tena-Colunga and Abrams, 1992a). Dynamic in-plane and out-of-plane stress analyses are addressed. Simplified 3-D stress evaluation can also be performed. The complexity of the analytical models to use in conjunction with the discrete models depends on the necessities, experience and judgment of each engineer, that is, they are not fixed by the proposed methodology

3. BASE ANALYTICAL MODEL FOR THE PROPOSED METHODOLOGY : MDOF DISCRETE DYNAMIC MODEL

The proposed methodology for the structural evaluation of low-rise masonry structures with flexible diaphragms when subjected to seismic loading is entirely based upon the discrete multi-degree-of-freedom (MDOF) dynamic model as outlined elsewhere (Tena-Colunga 1992a/b, Tena-Colunga and Abrams, 1992a). The discrete MDOF model was developed to simulate the dynamic response of low-rise masonry buildings with flexible diaphragms. This model represents the dynamic response of a structure in a given direction of interest by a reduced number of discrete masses associated to translational degrees of freedom acting in that direction. Flexible diaphragms are represented by elastic springs, which depending on the supporting conditions of the diaphragms are idealized as shear-bending or axial springs, or a combination of them (Fig 1). Walls resisting lateral forces in the direction under study are represented by equivalent condensed beams with translational degrees of freedom which include the rotations of the walls through static condensation. Soil-structure interaction effects can be easily considered in the modeling by defining an average lateral diminished matrix from the average lateral stiffness matrix and the foundation flexibility represented by two generalized springs (rotational and translational), as presented by Hjelmstad and Foutch (1988)

The discrete model have predicted well response time-histories of structural elements of instrumented masonry buildings during the Loma Prieta Earthquake, the firehouse of Gilroy and a two-story office building at Palo Alto, as it has been presented elsewhere (Tena-Colunga, 1992 a/b; Tena-Colunga and Abrams, 1992 a/b) Therefore, based upon the dynamic responses computed with the discrete models, different types of evaluation of particular structural components, or the structure as a whole, can be performed at specific times of interest In-plane and out-of-plane stress evaluations of masonry components can be done using the results computed from the discrete models together with a suitable stress model. A simplified method of 3-D dynamic analysis, named 3-D quasi-dynamic analysis (Tena-Colunga, 1992b, Tena-Colunga and Abrams 1992a, 1993 and 1994a), was developed based upon the dynamics of the discrete models. The case study of a low-rise masonry building with flexible diaphragms which obtained instrumented records from the 1989 Loma Prieta Earthquake, the firehouse of Gilroy, is presented to help illustrate the proposed methodology.

4. TWO-DIMENSIONAL IN-PLANE ANALYSIS

The discrete MDOF dynamic models can be used to identify peak dynamic responses (or responses at a given time-step of interest) in any structural element represented on them.

For example, peak accelerations for a given structural element (e.g., a wall) can be extracted from the dynamic analysis with the discrete models. These accelerations can be traduced to equivalent lateral forces. These equivalent lateral forces are used to estimate shear stresses within the element of interest, using from simple approaches (e.g., classic elastic solutions) to refined methods of analysis (e.g., finite elements)

4.1 Classic Elastic Solution For Shear Stresses

Quick estimates of in-plane shear stress can be obtained by employing the elastic solution given by Jourawski's formula :

$$\tau_{xy} = \frac{VQ}{Ib} \qquad \text{... (1)}$$

where τ_{xy} is the average shear stress through the width and at the depth of interest of a given cross section, V is the acting shear force, Q is the first moment of areas about the neutral axis of bending, I is the moment of inertia of the section with respect to the neutral axis, and b is the width of the cross section at the depth of interest. Shear stress is maximum at the neutral axis and null at the free edges. For a rectangular cross section, the maximum shear stress is 1.5 times the average shear stress, that is :

$$\tau_{max} = \frac{3V}{2A} \qquad \text{... (2)}$$

where A is the area of the cross section Equation 2 constitutes a crude approximation for a perforated wall because its cross section is not solid along its height. Nevertheless, it is a useful tool to obtain index values of peak shear stresses because it is simple.

4.2 Allowable Shear Stresses for Unreinforced Masonry

Computed shear stresses were compared against the allowable shear stresses of different masonry building codes and recommendations used in the United States, namely the ABK Methodology ("ABK", 1981), the Uniform Code for Building Conservation (UCBC; SEAOC, 1991), The 1991 Uniform Building Code (1991 UBC; ICBO, 1991) and the ACI 530-92 Code (TMS, 1992)

4.3 Classic Elastic Solution for Combined Bending and Axial Stresses

In unreinforced masonry elements, it is important to determine if a state of net tensile stress will exist because of the low tensile strength that URM has. Quick estimates of flexural stresses can be easily determined by using the classic combined stress equation :

$$f_a + f_b \leq F_a \qquad \text{... (3)}$$

where f_a is the acting axial stress, f_b is the acting bending stress, and F_a is an allowable working stress criterion given by a code. Equation 3 can be rewritten in different ways to fit the criteria of a particular building code.

4.4 Allowable Combined Stress Criteria

The UCBC Code specifies that URM piers need no to be analyzed for tensile stresses, thus, gives no recommendations to check for combined stress states. The ABK

Methodology has a capacity criterion that does not consider explicitly combined stress states in the evaluation of URM elements. The allowable combined stress criteria of the 1991 UBC and the ACI 530-92 codes is similar. The unity equation is the criterion for compressive stresses due to bending and axial load :

$$\frac{-f_a}{F_a}+\frac{f_b}{F_b}\leq 1 \qquad \text{... (4)}$$

where f_a is the acting axial stress (psi), f_b is the acting bending stress (psi), F_a is the allowable compressive axial stress in URM (psi), and F_b is the allowable compressive flexural stress in URM (psi). For net flexural tensile stresses due to bending and axial load , both codes adopt the following criterion :

$$-f_a+f_b\leq F_t \qquad \text{... (5)}$$

where F_t is the allowable tensile stress (psi), as addressed by each code (ACI Table 6.3.1.1 and UBC Sec. 2406 (c) 4). The allowable stresses of Eqs. 4 and 5 can be multiplied by 1.33 when lateral loading is considered in the analyses, as is the case of earthquake loading.

5. OUT-OF-PLANE ANALYSIS

The cracking or partial (total) collapse of walls due to the out-of-plane pushing of flexible diaphragms has been a dominant failure mode observed in URM structures during past earthquakes. Nevertheless, little attention has been devoted to develop suitable analytical methods to evaluate this phenomenon. The discrete MDOF dynamic models can be used to predict critical lateral displacements imposed to the walls orthogonal to the direction of analysis These predicted lateral displacements can be imposed on FEM representations of these orthogonal walls to evaluate if the horizontal bending stresses do not surpass an allowable tensile stress criterion The refinement of the FEM mesh will depend on the judgment and necessities of each individual. To the authors knowledge, there are no simplified methods available in building codes and basic literature (Schneider & Dickey, 1987) to evaluate stresses for URM structures when subjected to out-of-plane pushing of flexible diaphragms due to lateral (earthquake) loading Thus, researchers, building officials and practicing engineers should sum efforts in this direction in order to develop suitable simplified methods which could be easily used and understood by anyone interested on the subject.

6 SIMPLIFIED 3-D DYNAMIC ANALYSIS

A simplified method of 3-D dynamic analysis, named 3-D quasi-dynamic analysis, which is based entirely on the MDOF discrete models, has been developed and primarily used to evaluate stress states at identified times of peak dynamic responses of structures with flexible diaphragms. The quasi-dynamic analysis has been presented in detail elsewhere (Tena-Colunga, 1992b; Tena-Colunga and Abrams, 1992a, 1993 and 1994a). The quasi-dynamic analysis consists of extracting the accelerations predicted by the time-step integration analyses of two 2-D discrete MDOF dynamic models of a given structure (each discrete model corresponding to one of the principal orthogonal directions of the structure) at given times of interest. These accelerations are imposed on a 3-D representation of the structure (for example, a finite element mesh) as equivalent static forces Accelerations at the diaphragms are assumed to act over the same tributary areas of the diaphragms considered in the 2-D discrete dynamic models. This method was compared to the more traditional 3-D modal time-step integration and had a general good agreement with it, using considerably less computational effort.

7. CASE STUDY : OLD FIREHOUSE AT GILROY

The building is a two-story historic building located in downtown Gilroy, California. The southeast view of the firehouse is depicted in Fig. 2. Gilroy is located approximately 15 miles south east of the epicenter of the Loma Prieta earthquake. The structure was built in 1890 and is a box-type structure, where the lateral force resisting system is composed of unreinforced masonry brick walls together with flexible diaphragms (wood sheathing) floor systems (Fig. 3). The plywood diaphragms consist of 2.54 cm (1") by 10.12 cm (4") timbers running in the diagonal direction and nailed to timber joists that are supported by built-up trusses. The plywood thickness is 1.27 cm (0.5") for all spans with the exception of the south diaphragm at the first floor, which is 1.59 cm (0.625") thick. All walls are three-wythe, 30.54 cm (12") running-bond unreinforced masonry brick walls joined by mortar bed joints 1.27 cm (0.5") thick, except for the south (front view) wall, which thickness varies from 30 54 cm (12") at the openings zone to 40.64 cm (16") and 43.18 cm (17") for the exterior piers as a result of architectural considerations. The diaphragms and the walls were tied by 1.91 cm (0.75") ϕ steel rods anchored in the outside wythe of the walls by a hook, and with or without a hook in the diaphragms. The building is founded on spread footings.

Prior to the Loma Prieta Earthquake, the former firehouse was instrumented by California Strong Motion Instrumentation Program (CSMIP) with six accelerometers (Fig. 3) Recorded peak ground accelerations were as high as 0.29g (sensor 3, Figs. 3) and peak roof accelerations as high as 0.79g at the diaphragm and 0 41g at the central wall (sensors 5 and 4, same figure) Considerable amplifications of the peak accelerations between the ground and the roof, and between the walls and the diaphragms were observed The former firehouse withstood the Loma Prieta Earthquake with little damage The structure has been the subject of a detailed investigation which results can be consulted elsewhere (Tena-Colunga, 1992b, Tena-Colunga and Abrams, 1992a).

7.1 Discrete Models

The discrete models presented on Fig. 1 were used to predict peak accelerations on the walls of the firehouse in the E-W and N-S directions. The predicted peak accelerations for the E-W direction are illustrated in Fig 4 It can be observed that the walls with more openings (e g. south wall) experience higher accelerations than the more solid walls (e.g. north wall) as a consequence of the diaphragm flexibility, as it has been pointed out in previous works (Tena-Colunga, 1992b; Tena-Colunga and Abrams, 1992a and 1994b). With these accelerations and the tributary masses, equivalent lateral forces were computed.

7.2 2-D In-Plane Shear Analyses

Two different methods were used to evaluate in-plane shear stresses. Quick estimates of in-plane shear stresses were obtained using classic elastic solutions (Eqs. 1 and 2), and they are compared against the ultimate an allowable shear stresses given by different masonry codes and recommendations of the US in Table 1. The compressive strength of the masonry, f_m was taken as 9.5 MPa (1325 psi), according to the lab material tests performed with the bricks claimed from the firehouse, and the basic bed-joint shear stress v_t was defined as 0 61 MPa (85 psi) from measured in-place shear data (Tena-Colunga, 1992b; Tena-Colunga and Abrams, 1992a). The amount of compressive axial stress for each wall due to dead load only is also given in Table 1.

The maximum shear stresses presented in Table 1 are much higher than the allowable shear limits prescribed by the UCBC code provisions, especially for the south, central and east walls. Hence, under this criterion, severe shear cracking should be expected at these walls. In contrast, the structure stood the Loma Prieta Earthquake with little damage (at the south wall only), as described in previous works (Tena-Colunga, 1992b;

Tena-Colunga and Abrams, 1992a). The UCBC provisions constitute the current state-of-the-art in the evaluation of existing URM buildings.

Table 1. Estimated vs code shear stresses at the firehouse of Gilroy. Units : MPa (psi)

Wall	Estimated Wall Stresses			Ultimate Shear	Allowable Shear		
	Axial	Ave Shear	Max Shear	ABK (max)	UCBC (max)	UBC (ave)	ACI (ave)
South	0.32 (44.)	0.29 (40.)	0.43 (60.)	0.58 (81.)	0.11 (15.)	0.14 (20.)	0.39 (55.)
Central	0.29 (40.)	0.24 (33.)	0.35 (49.)	0.56 (78.)	0.11 (15.)	0.14 (19.)	0.39 (55.)
North	0.18 (25.)	0.08 (11.)	0.12 (16.)	0.48 (67.)	0.09 (12.)	0.11 (16.)	0.39 (55.)
East	0.29 (41.)	0.16 (22.)	0 24 (34.)	0.57 (79.)	0.11 (15.)	0.14 (19.)	0.39 (55.)
West	0.21 (29.)	0.08 (11)	0.11 (16.)	0.50 (70.)	0.09 (13.)	0.12 (17.)	0.39 (55.)

On the other hand, data in Table 1 suggest that, for all walls, the computed maximum shear stresses did not surpass the ultimate shear stress criterion of the ABK Methodology. Maximum shear stresses were less than 74% of the determined in-plane shear strength. On the basis of approximate stresses computed this way, it is credible that masonry should have not cracked during the Loma Prieta Earthquake.

Under the 1991 UBC criterion, only the north and west walls should be expected to survive the earthquake without severe damage, whereas under the ACI 530-92 criterion, only the south wall should have experienced shear damage while the rest of the walls should have remained undamaged. Therefore, data in Table 1 suggests that the UBC criterion is very conservative for regarding allowable shear stresses for URM bearing/shear walls, whereas the provisions of the ACI 530-92 code seem to apply well in this particular case study

More refined stress analyses were performed using 2-D finite elements. All the walls were modeled using eight-noded, two-DOF per node quadratic serendipity isoparametric plane stress elements available in the finite element program POLO-FINITE (Lopez et al, 1987). For the illustration purposes of the present work, only the analysis of the critical wall, the south wall (Fig. 5) is addressed. Mesh refinement attended to research purposes, specially to define qualitative stress patterns. For practical purposes, a coarser mesh could have been used. The variations in the thickness of the wall due to architectural aesthetics were included in the modeling. Young's modulus of the masonry was taken as 515 ksi, according to the data obtained from the test of the prisms made with reclaimed material from the firehouse (Tena-Colunga, 1992b, Tena-Colunga and Abrams, 1992a). Equivalent seismic forces obtained from the accelerations of Fig. 4 were uniformly distributed along the edges of the elements that define the floor levels. Self weight of the masonry was introduced in all elements through a uniformly distributed body force Additional gravitational loads were also applied along the edges of the elements that define the floor levels. Because of the symmetry of the wall with respect to its vertical axis, the direction of the seismic load was not a major factor to consider in the stress analysis. However, the stress contours presented in Fig. 5 correspond to a lateral loading acting from west to east (Fig. 4). A mirror image stress contours should be present if the seismic loading would have been considered to act from east to west.

Shear stress contours are presented in Fig. 5. Eight contour levels were selected to evaluate the overall response of the wall according to different code provisions. Target values that define contour boundaries correspond to those given by the UCBC, the ACI 530-88, and the ABK Methodology, according to Table 1. An upper target value of 0.72 MPa (100 psi) was introduced to identify regions where shear stresses were predicted to surpass the ultimate maximum shear capacity of the masonry, according to the ABK Methodology.

Stress contours presented suggest that under the critical state of loading, the ultimate shear capacity (81 psi) could have been surpassed primarily in a region between the extreme outer upper corners of the windows at the first floor and the outer windows of the second floor. A mild shear crack was observed at that location in the west side of the wall. The contours also identify high shear stress concentrations in the upper corners of the outer windows at the second floor. Given the unrefinement of the mesh in the regions nearby openings, this information should be ignored. In general, stress contours suggest that the wall should have experienced some damage in a relatively small wall extension, according to the ultimate shear capacity criterion of the ABK Methodology. The wall was lightly damaged in one of the critical regions defined by the contours under the ABK Methodology criterion.

Under the ACI 530-92 code guidelines (allowable shear stress of 55 psi), the stress contours of Fig. 5 define a larger affected area in the critical regions identified under the ABK criterion. In addition, the suspicious damaged zones spread into the first story piers and to regions nearby the top of the central door opening and inner upper corner of the wide window openings at the first floor. Some mild cracking was observed in the last region, however, no damage was observed at the first story piers during the field survey

If the maximum allowable shear stress of 15 psi of the UCBC is considered, then, stress contours of Fig. 5 suggest that the walls could have experienced generalized and extensive shear damage According to the damage observed during the field survey of the firehouse, it is felt that the shear stress criterion of the UCBC code is very conservative. The south wall presented a negligible amount of shear damage whereas the UCBC code predicted severe generalized damage The ultimate shear strength criterion of the ABK Methodology has given a reasonable prediction of the observed damage for this case study The ACI provisions, although intended for design of new construction rather than for evaluation of old structures, gave a reasonable prediction of the observed damage at the wall also.

7.3 2-D In-Plane Combined Stress Analyses

Estimated net flexural tensile and compressive stresses for each wall using crude methods (Eqs 4 and 5), and associated to the lateral loads of Fig. 4, are summarized and compared against the criteria of the 1991 UBC and ACI 530-92 codes in Table 2. The allowable flexural tensile stress, F_t , has been already increased by 33 percent in the Table. The ACI 530-92 code allows no tensile stress for this condition. Computed values from the left hand side terms of Eqs 4 and 5, are presented in the table to compare more easily against code's criteria In Table 2, the axial compressive stress due to dead load only is defined by f_{ad}, while the compressive stress due to dead and live load is defined by f_{adl}.

It could be observed from Table 2 that, under the critical loading condition, all walls should have no problem for net compressive stresses (Eq 4). All walls but the west wall are subjected to a net tensile stress state under the critical condition (Eq 5). The central wall is the only one which could have experienced net flexural stresses higher than those allowed by the UBC code. However, no flexural tensile cracks were observed in any of the walls at the first story piers. The only cracking caused by tensile stresses in the building was the horizontal bending cracks at the bottom and the top of the slender piers among window openings at the second level of the south wall. Therefore, this crude method of evaluation correlates well with which was observed in the structure.

Table 2. Evaluation of estimated combined stress states with respect to the 1991 UBC and ACI 530-92 codes

Wall	Estimated stresses (psi)			Code allowable stresses (psi)					Combined stress criteria		
	f_b	f_{ad}	f_{adl}	F_b	F_a UBC	F_a ACI	F_t UBC	F_t ACI	Eq. 5 (psi)	Eq. 4 UBC	Eq. 4 ACI
South	68	44.	55.	437.	265.	331.	36	0.	24	0.36	0.32
Central	99	40	53.	437.	265.	331.	36.	0.	59	0.43	0.39
North	33.	25.	30.	437.	265.	331.	36.	0.	8	0.19	0.17
East	55.	41.	50.	437.	265.	331.	36.	0.	14	0.31	0.28
West	22.	29.	35.	437.	265.	331	36.	0.	-7	0.18	0.16

More refined stress analyses were performed using the 2-D finite elements, with the elements characteristics and details as described before. For the illustration purposes of the present work, only the analysis of the critical wall, the central wall (Fig. 6) is addressed. Critical stress contours were defined when lateral loading was applied from west to east (left to right in Fig. 6). Target values were chosen to identify net tensile stresses and some allowable tensile and compressive stresses according to code criteria (Table 2). It can be observed from the stress contours of Fig. 6 that under the critical state of loading, some net tensile stresses (ACI 530-92 criterion) are identified primarily at opposite corners of the door opening (left-top and right-bottom corners), at the second floor level and the left side of the left and right piers at the first story If the 1991 UBC allowable tensile stress normal to the bed joints of 0.26 MPa (36 psi) is considered, the identified regions are less spread. The higher tensile stresses are concentrated at the corners and they should be disregarded because the mesh is too coarse in those zones to accurately estimate stresses. The central wall was primarily under compression and within the allowable limits

7.4 Out-Of-Plane Stress Analysis

Horizontal normal stresses caused by the out-of-plane pushing action of the diaphragms and the walls perpendicular to the wall under consideration were studied using finite elements. The case study of the south wall is presented for illustration purposes for out-of-plane bending because is the weaker wall of the firehouse in the E-W direction. Standard isoparametric quadratic shell elements (eight nodes, six-dof-per node) were used to determine out-of-plane stress distributions instead of the more traditional plate bending elements, because of the irregularities in the geometry of the walls and openings which requires trapezoidal elements in the mesh. The soil-foundation system was modeled with rigid beam elements connected to vertical and horizontal springs in the direction of the out-of-plane displacements. The stiffness of the spring elements was defined according to the ATC 3-06 specifications (ATC, 1978).

Out-of-plane stresses were determined by imposing to the wall under study the corrected predicted in-plane displacements obtained from the discrete models at the center of the diaphragms and at the wall's running in the perpendicular direction at the time where the peak displacement was obtained Therefore, for the south wall, the predicted and corrected displacements obtained from the discrete model in the N-S direction (Fig. 1) at t=5.45 seconds were imposed to the nodes which represent the intersection of the discrete masses of the N-S modeling in the south wall. These displacements are summarized in Table 3

The computed deflected out-of-plane shape of the south wall under these imposed deformations is presented in Fig. 7, where a frame of reference is offered to help visualize the out-of-plane deformations. The displacements at the base are uniform because the foundation was assumed to behave as a rigid body. The horizontal normal stress contours obtained from the out-of-plane finite element analysis under these imposed deformations are presented in Fig. 8. Contour values were selected to highlight net tensile stress states. The 1991 UBC Code specifies an allowable tensile stress of 0.52MPa (72 psi) for flexural tension normal to the head joints. Tensile stresses exceeding the UBC allowable stresses were located at the center of the wall below the central window opening at the second floor. The highest tensile stresses were found nearby the intersection of the wall with the center of the diaphragm at the first story. The identify over stressed region was narrow. No cracking under this condition was observed in the wall. Although most of the central part of the wall was subjected to tensile stresses normal to the head joints, the magnitude of these stresses was generally lower than the allowable stress prescribed by the UBC code. Therefore, the south wall was expected to present little or no damage under out-of-plane pushing. An effective connection between the walls and the diaphragms may have contributed to an adequate performance of the walls under out-of-plane loading.

Table 3. Imposed and corrected out-of-plane displacements to the south wall at t=5.45 seconds. Units : cm (inches)

Intersection with	First Floor	Roof
East Wall	0.23 (0.09)	0.36 (0.14)
Center of South Diaphragm	0.79 (0.31)	1.02 (0.40)
West Wall	0.18 (0.07)	0.25 (0.10)

8. SUMMARY AND CONCLUSIONS

A comprehensive methodology for the structural evaluation of low-rise masonry structures with flexible diaphragms when subjected to moderate or strong earthquake motions based on discrete MDOF dynamic models was presented. Different types of evaluation of structural components, or the structure as a whole, were addressed using both simplified and elaborated procedures in conjunction with the discrete models. In-plane and out-of-plane stress evaluations of masonry components were illustrated using the results computed from the discrete models together with a suitable model. The case study of a low-rise masonry building with flexible diaphragms which obtained instrumented records from the 1989 Loma Prieta Earthquake was presented to help illustrate the proposed methodology. The proposed evaluation procedure was helpful to correlate the predicted responses against the observed extent of damage after the earthquake. The methodology was helpful to evaluate another instrumented low-rise masonry building (Tena-Colunga and Abrams, 1992b). It is felt that the proposed methodology based on the discrete models can be used with confidence in the seismic evaluation of low-rise masonry structures with flexible diaphragms. The present study also highlights the need to develop simplified methods to evaluate out-of-plane stresses in wall components due to the pushing action of flexible diaphragms, as this is a dominant failure mode observed during recent earthquakes

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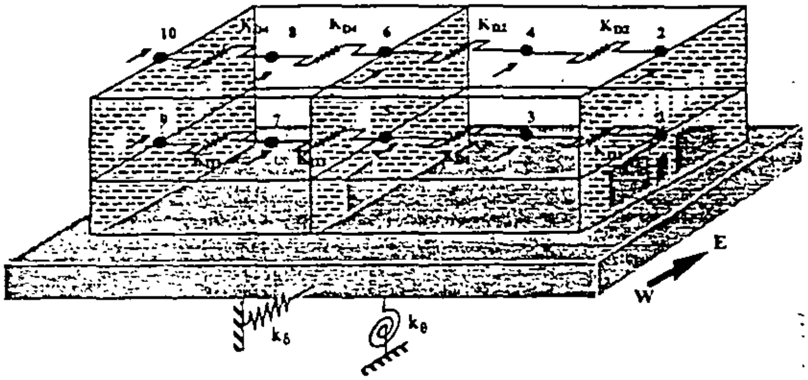
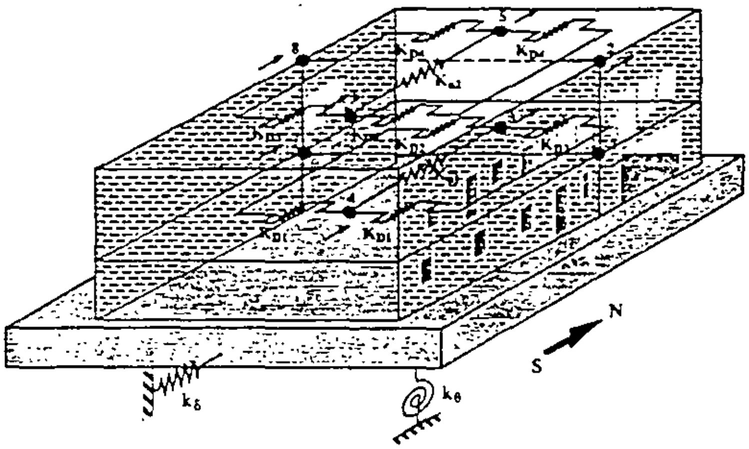


E-W direction



N-S direction

Figure 1 Discrete dynamic models for the firehouse of Gilroy

Figure 2. Southeast view of the firehouse of Gilroy

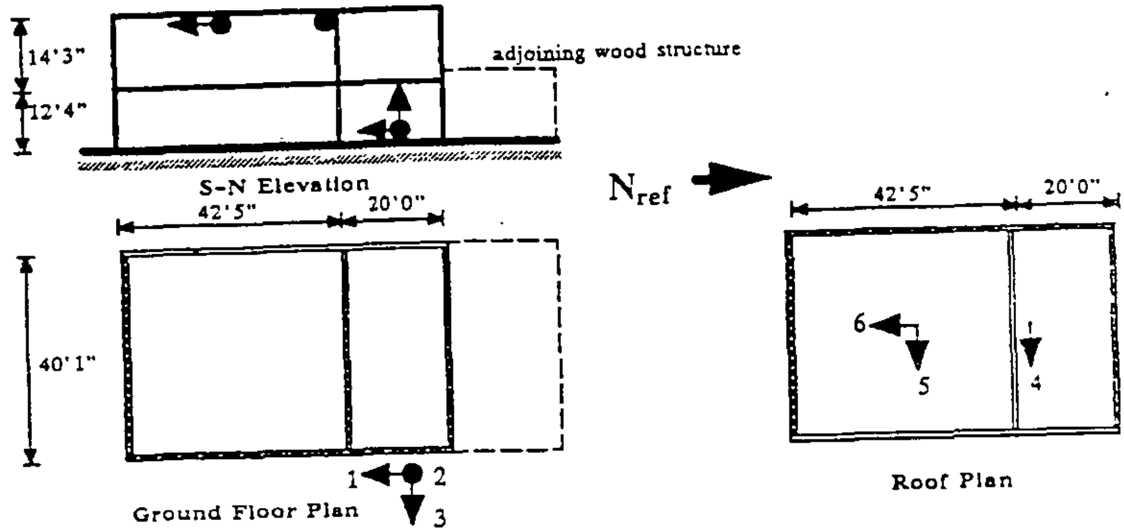


Figure 3. Plan views and location of sensor at the firehouse of Gilroy

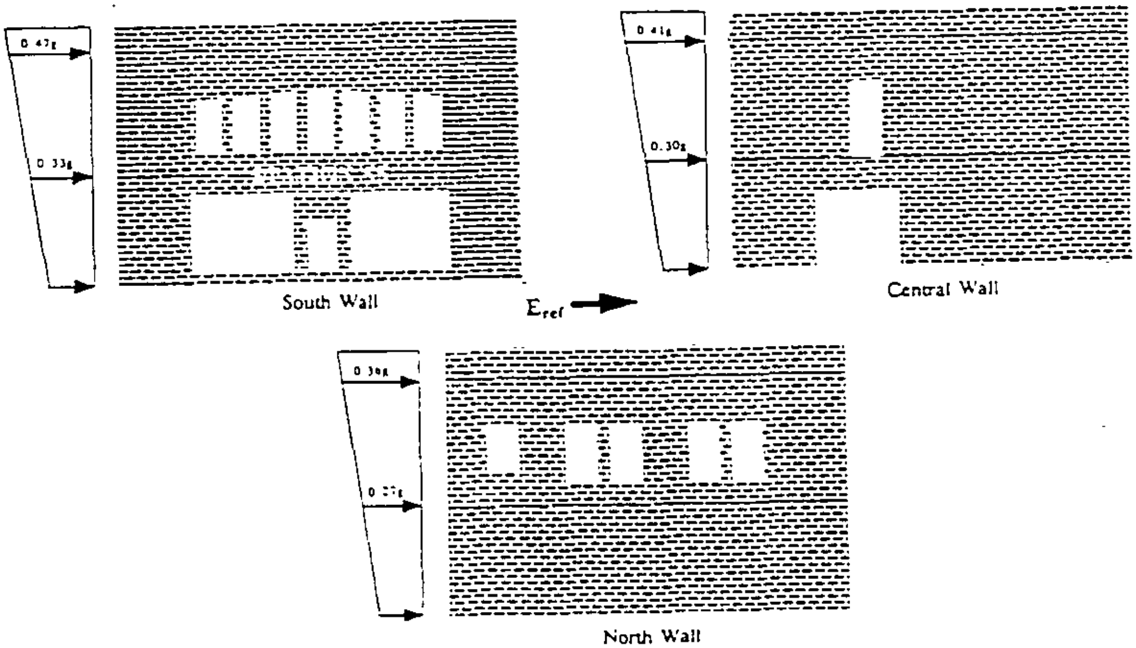


Figure.4. Lateral acceleration distributions for the E-W running walls

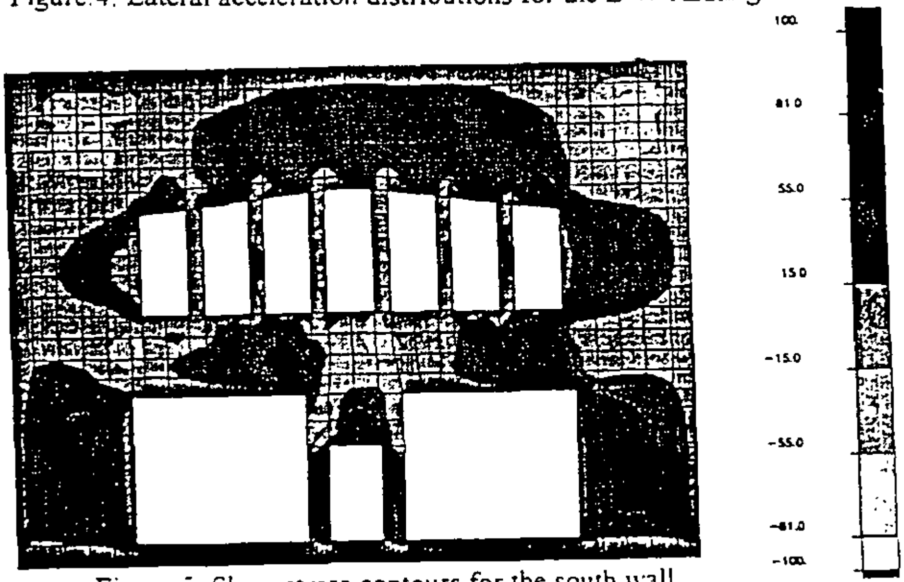


Figure 5. Shear stress contours for the south wall

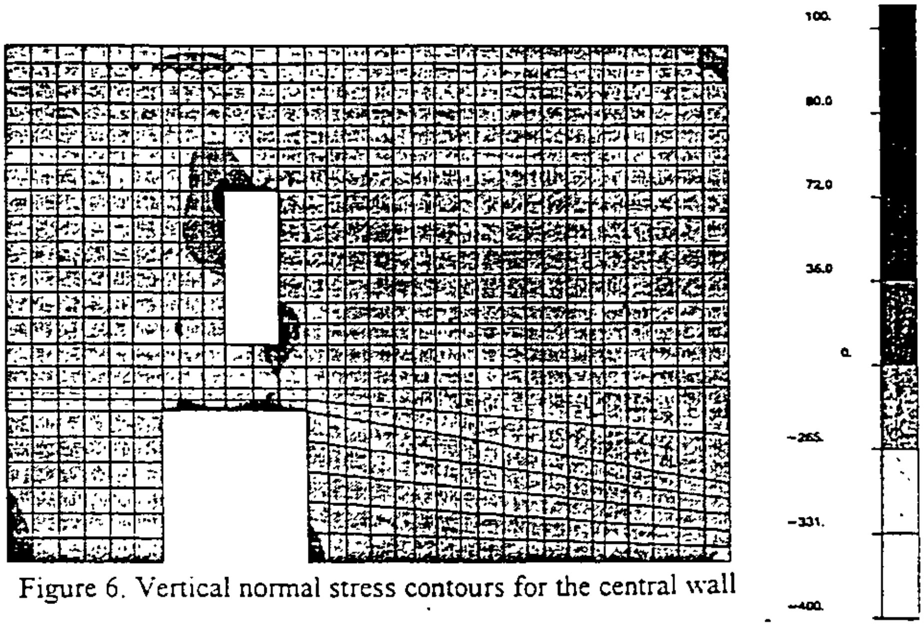


Figure 6. Vertical normal stress contours for the central wall

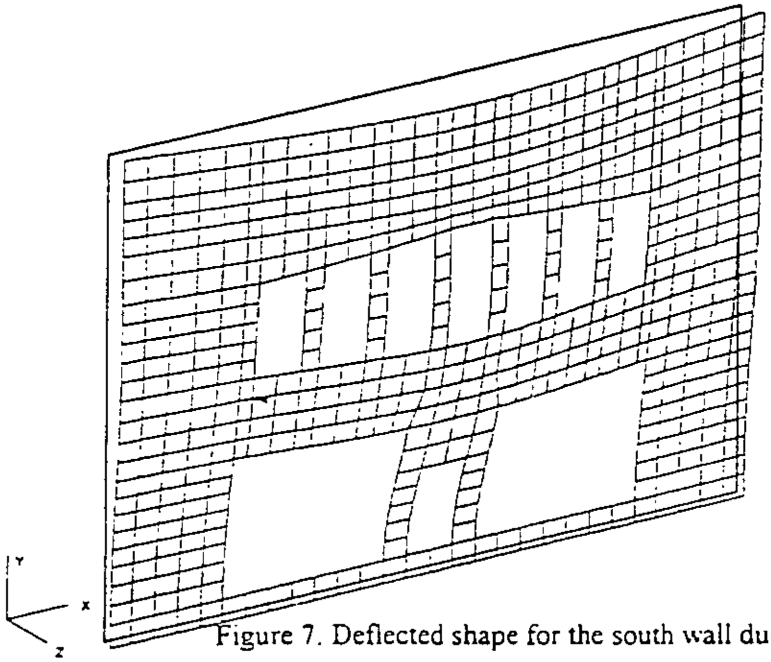


Figure 7. Deflected shape for the south wall due to out-of-plane action

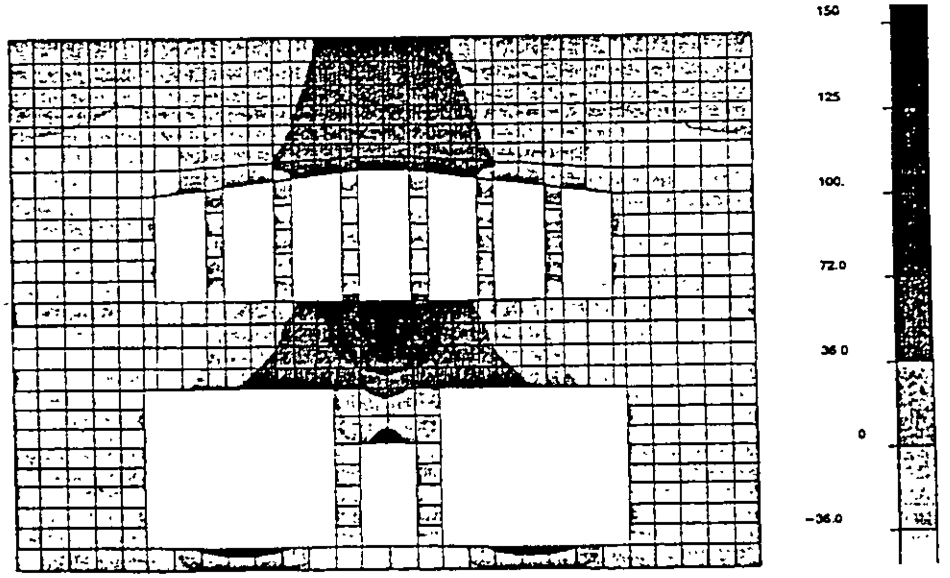


Figure 8. Horizonal normal stress contours for the south wall due to out-of-plane action

FACULTAD DE INGENIERIA U.N.A.M.
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CURSOS ABIERTOS

XXVI CURSO INTERNACIONAL DE INGENIERIA SISMICA

MÓDULO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

SEISMIC BEHAVIOR OF STRUCTURES WITH FLEXIBLE DIAPHRAGMS

EXPOSITOR: DR. ARTURO TENA COLUNGA
PALACIO DE MINERIA
SEPTIEMBRE DE 2000

SEISMIC BEHAVIOR OF STRUCTURES WITH FLEXIBLE DIAPHRAGMS

By Arturo Tena-Colunga[1] and Daniel P. Abrams,[2] Members, ASCE

ABSTRACT: The influence of floor flexibility on the seismic response of building structures is discussed through comparison of the computed seismic response for structures with flexible diaphragms and counterpart structures with rigid diaphragms. Case studies of three existing buildings with flexible diaphragms and analogous systems with rigid diaphragms are presented to illustrate these differences. Each building was subjected to the 1989 Loma Prieta Earthquake. The structures were: (1) A two-story firehouse in Gilroy with unreinforced masonry walls; (2) a two-story timber office building in Palo Alto with grouted and reinforced clay-unit masonry shear walls; and (3) an eight-story hotel in Oakland with unreinforced clay-unit masonry and reinforced-concrete shear walls. The analytical studies show that, in some cases, diaphragm and shear-wall accelerations can increase with the flexibility of the diaphragm. Torsional forces can reduce considerably as diaphragm flexibility increases. Further, approximate expressions prescribed in current seismic codes can underestimate the period of vibration of systems with flexible diaphragms.

INTRODUCTION

Structures with flexible floor systems behave differently under dynamic lateral loading than structures with rigid diaphragms. The American Plywood Association (*APA* 1983) recognizes this fact by publishing a design guide that discusses procedures for determining and distributing seismic forces using flexible plywood diaphragms. The ABK Methodology ("Methodology" 1984) recommends analytical procedures for the horizontal displacement control of flexible diaphragms that are based on dynamic testing and modeling. Recommendations for the seismic evaluation of URM buildings with flexible diaphragms contained in the Uniform Code for Building Conservation ("Seismic" 1994) were derived from the ABK research as were similar provisions included in *FEMA 178* ("Evaluation" 1992). The *Uniform Building Code* (UBC) (1994) contains provisions for the seismic design of newly constructed building systems that consider flexible-diaphragm behavior. In Section 1628.5, a statement is made that story shear force shall be distributed to various elements in proportion to their relative rigidities, considering the rigidity of the diaphragm. Diaphragms are to be considered flexible when the diaphragm deflection exceeds twice the story drift. Torsional effects are addressed in UBC Section 1628.6 only for the case when diaphragms are not flexible. Diaphragm deflections shall not exceed permissible deflections of attached elements per Section 1631.2.9.

The concept that building structures with flexible diaphragms behave differently from systems with rigid diaphragms is fundamental. However, flexible-diaphragm systems are still analyzed with criteria and recommendations developed for structures with rigid diaphragms. This practice is not necessarily conservative. As discussed in this paper, structures with flexible diaphragms can experience higher accelerations and displacements than structures with rigid diaphragms, and their fundamental periods of vibration can be significantly longer.

In moderate and strong earthquakes, a number of timber diaphragms may be forced to develop their full strength as they are deformed past the proportional limit. When this happens, dynamic response will be nonlinear as the diaphragms tend to degrade in stiffness. In such case, the lateral force applied to a shear wall will not exceed the strength of the diaphragm, and it is feasible to simply apply lateral forces to a wall that are equal to the diaphragm strength (as prescribed by the ABK method). Because diaphragm damage was not observed in the three subject buildings described in the following sections, diaphragms were modeled with linear elements. Conclusions drawn from this study are applicable only to elastic diaphragms.

SUBJECT BUILDINGS

Three existing buildings, which were subjected to the 1989 Loma Prieta Earthquake were selected for analytical investigation. Each building system consisted of lateral-force-resisting elements of masonry and flexible floor and roof diaphragms of timber. The seismic response of each building system is computed and compared with measured accelerations when available. The response of analogous, hypothetical systems with rigid diaphragms are then computed to contrast the response of actual systems with flexible diaphragms.

Firehouse in Gilroy

The first subject building is a two-story former firehouse, located in Gilroy, California (Fig. 1). The town of Gilroy is approximately 25 km south east of Loma Prieta. The structure was built in 1892 and is one of the first instrumented unreinforced masonry buildings to be subjected to a moderate earthquake. The firehouse is a box-type structure, where the lateral-force-resisting elements are unreinforced masonry brick walls tied together with flexible timber floor and roof diaphragms (Fig. 2). The building was rehabilitated before the 1989 earth-

FIG. 1. South Elevation of Gilroy Firehouse

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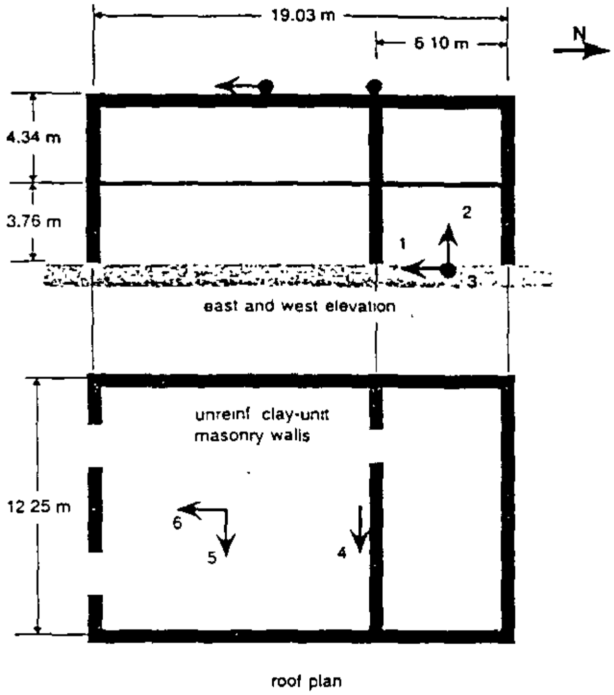

FIG. 2. Accelerometer Locations for Gilroy Firehouse

quake by adding plywood sheets to diagonal sheating (25 × 101 mm). The plywood thickness is 13 mm for all spans except the south diaphragm at the first floor, which is 16-mm thick.

All unreinforced masonry walls are three wythes of clay-unit brick laid in running bond with header units at every fifth course. Typical wall thickness is 305 mm. The thickness of the south wall varies from 305 mm around the window openings to 406 mm and 432 mm at the exterior piers. Mortar is in good condition, and was made with sand, hydrated lime, and some cement. Bed and head joints are 12-mm thick. The floor, ceiling, and roof diaphragms are tied to the masonry walls with 19-mm diameter steel rods that are anchored in the exterior wythes with a hook. The building is founded on spread footings.

The building was instrumented by the California Strong Motion Instrumentation Program (CSMIP) with six accelerometers (Shakal et al. 1989). During the Loma Prieta Earthquake, recorded ground accelerations in the east-west direction were as high as 0.29*g* (sensor 3, Fig. 2). East-west accelerations were as high as 0.79*g* at the midspan of the roof diaphragm and 0.41*g* at the top of the central wall (sensors 5 and 4, respectively). The structure withstood the earthquake with little damage for reasons discussed in Tena-Colunga and Abrams (1992).

Office Building in Palo Alto

The second subject building is a two-story office building (Fig. 3) in Palo Alto, California (50 km north of Loma Prieta), which was built in 1974. The lateral-force-resisting system consists of two grouted and reinforced, clay-unit masonry walls at the north and south ends. Flexible floor and roof diaphragms span between each wall (Fig. 4). The second floor consists of 38-mm-thick lightweight concrete over 19-mm-thick Douglas Fir plywood mounted on 914-mm open truss joists running in the east-west direction every 610 mm. Two interior glulam beams (130 × 381 mm) running in the north-south direction and four exterior glulam beams (130 × 419 mm) complete the floor system. The roof diaphragm is more flexible than the first-floor diaphragm, with 13-mm-thick plywood and deep interior and exterior timber beams running in both directions. The aspect ratio of both diaphragms (length/width) is 1.87.

FIG. 3. Southwest Elevation of Palo Alto Office Building [from Shakal et al. (1989)]

The L-shaped masonry walls are 305-mm thick. Grout has been placed within a 178-mm cavity between the two wythes. Clay-masonry units are grade MW and are laid in running bond with a type S mortar. Walls are reinforced with No. 4 bars at 305 mm in both horizontal and vertical directions. Additional vertical reinforcing bars were provided at the corners. All reinforcement steel was specified to be Grade 40, and the length of lap splices was specified at 50-bar diameters. Diaphragms are tied to the walls at ledgers with 19-mm-diameter steel rods that are anchored with a hook to the grouted collar joint. These ledgers are nominally placed every 610 mm in both directions. Plywood is connected to the ledgers with n

The gravity-load system consists of tubular steel colu (89-mm diameter) and exterior glulam columns. The building is founded on spread footings.

The building was instrumented by CSMIP with seven accelerometers. Recorded ground accelerations during the Loma Prieta Earthquake were as high as 0.21*g* (sensor 3, Fig. 4), and roof accelerations were as high as 0.34*g* at the north wall and 0.55*g* at the midspan of the diaphragm (sensors 4 and 5, Fig. 4). The office building sustained the earthquake with no

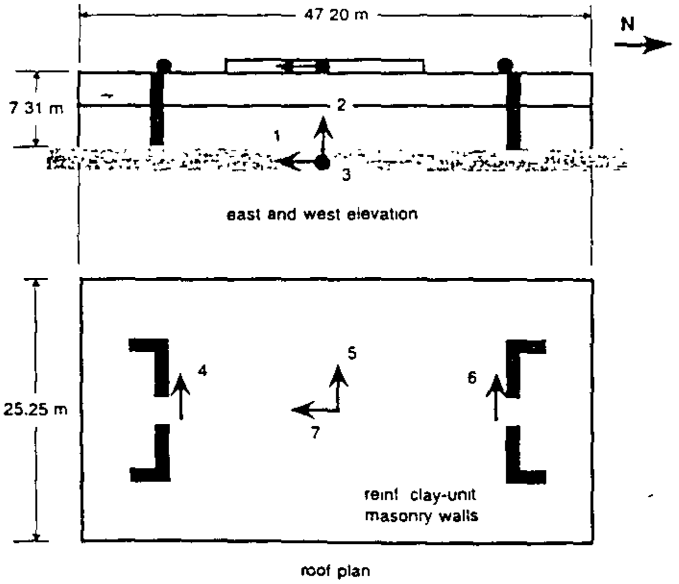

FIG. 4. Accelerometer Locations for Palo Alto Office Building

damage. The structure has been the subject of a detailed investigation described in Tena-Colunga and Abrams (1992c).

Hotel in Oakland

The Tourraine Hotel (Fig. 5) is a seven-story building located in downtown Oakland, California (76 km north of Loma Prieta) and constructed in 1914. The plan of the first floor is rectangular in shape, whereas the plan of the second through seventh floors is C-shaped (Fig. 5). The building is 30.5-m long on the north elevation (Fig. 6), 18.3-m wide on the east elevation, and 28.1-m tall.

The lateral load system consists of unreinforced masonry and lightweight reinforced-concrete shear walls with flexible wood diaphragms. The east and north facade walls are 356-mm-thick clay-unit masonry walls, whereas the south and west property-line walls are 203-mm-thick lightweight concrete walls with brick veneer. The vertical load-carrying system consists of steel frames with semirigid connections encased into the masonry and reinforced-concrete walls. All steel connections are riveted and consist of clip angles at the top and bottom of beams for connections to columns.

Floor systems are 19-mm-thick plywood diaphragms, with rough timber floor joists (76 × 330 mm) supported on steel beams. Typical floors have 25 × 102 mm straight sheathing overlaid by 25 × 76 mm tongue and groove flooring. The second floor is sheathed with 25 × 152 mm diagonal sheathing. "Government" anchors were used to connect the floor systems to the exterior masonry walls. Two anchors per bay were typically used. Government anchors were also used as an extra tie between floor joists over interior beams. Ceilings are cement plaster on expanded metal lath attached directly to the underside of the ceiling joists. Typical partition-wall construction is plaster on metal lath over wood studs.

Moderate structural damage was observed following the Loma Prieta Earthquake. Diagonal shear cracks were observed at the second story piers of both the east and the north masonry walls, as well as between window openings on the south reinforced-concrete walls at the first and second stories. Extensive cracking was seen on the first-story piers encasing the steel columns, in particular at the northeastern corner of the building. Extensive damage was observed to the plaster on the first-floor ceiling. All floor systems were found in good condition after the earthquake. There was no evidence of foundation distress.

FIG. 5. Southeast Elevation of Tourraine Hotel

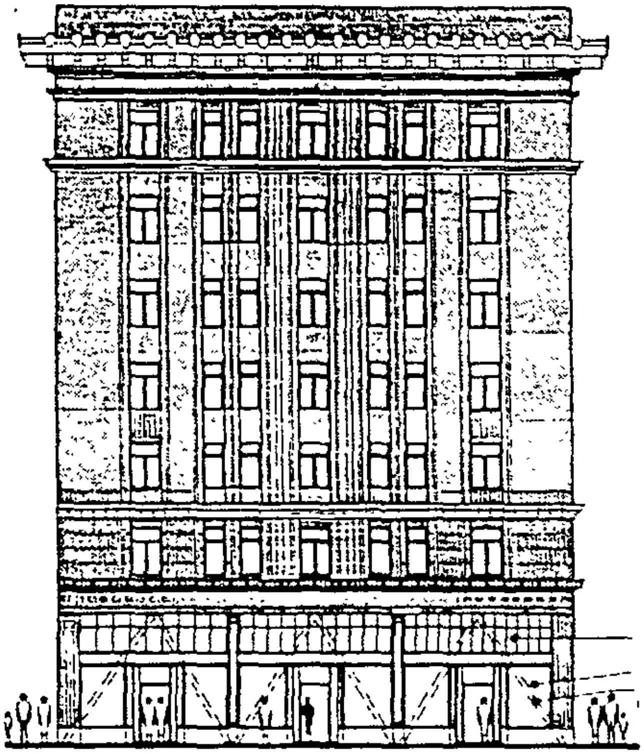

FIG. 6. North Elevation of Tourraine Hotel (Courtesy of URS Consultants)

Observed damage can be associated with the torsional flexibility of the building on the first story. North and east elevations consist of masonry facade walls that are discontinuous on the first story while south and west elevations consist of solid walls of reinforced concrete. The large eccentricity of the lateral forces about the center of stiffness resulted in substantial torsional forces, and thus damage to the corner elements.

ANALYTICAL STUDIES

Discrete, multidegree-of-freedom (MDOF) dynamic models were developed to study the measured dynamic response of the Gilroy firehouse and the Palo Alto office building. The MDOF dynamic model (Tena-Colunga 1992; Tena-Colunga and Abrams 1992) was based on linear masonry behavior and flexible and linear diaphragms. Soil flexibility was modeled with translational and rotational springs attached to the foundation. The 10-DOF and eight-DOF discrete models in Fig. 7 were used to represent the dynamic response of the firehouse in the east-west and north-south directions, respectively, whereas the six-DOF discrete models in Fig. 8 were used to model the dynamic response of the office building at Palo Alto in each direction. Acceleration and displacement histories were computed with these models and compared favorably against the measured seismic response in earlier studies. Then, models were used to estimate the dynamic response of the same building systems with rigid diaphragms so that correlations could be made between the two cases.

The seismic response of each of the three buildings was examined with respect to: (1) maximum lateral accelerations; (2) maximum lateral displacements; (3) torsional effects; and (4) natural periods.

Maximum Lateral Accelerations

Computed peak accelerations for flexible and rigid diaphragm systems are shown in Table 1 for the firehouse and in Table 2 for the office building.

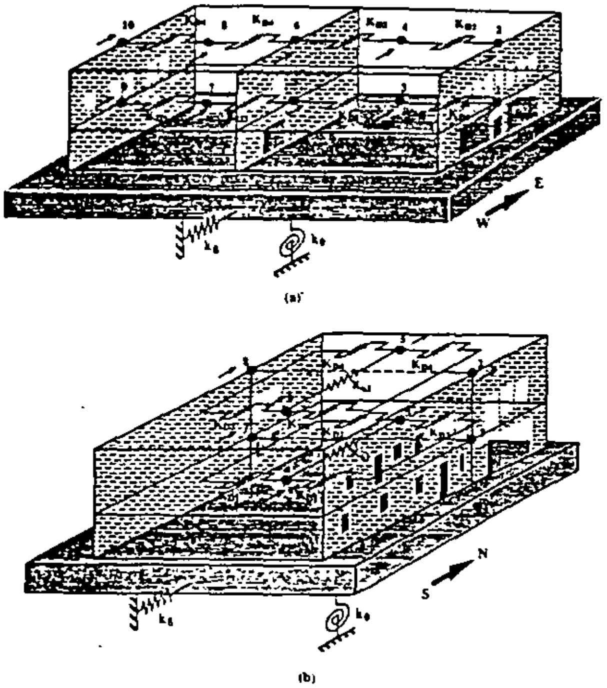

FIG. 7. Discrete Dynamic Models for Firehouse: (a) East-West Direction; (b) North-South Direction

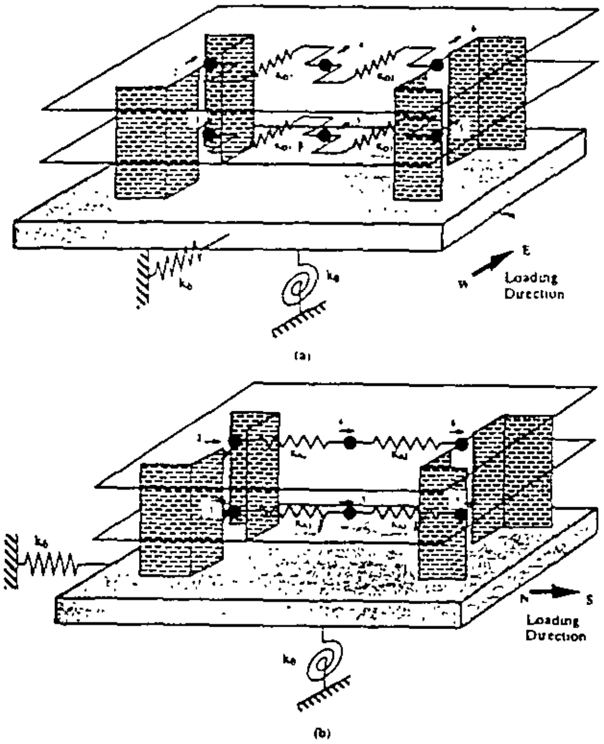

FIG. 8. Discrete Dynamic Models for Office Building: (a) East-West Direction; (b) North-South Direction

For a fixed-base condition, the assumption of flexible diaphragms can result in lower wall accelerations than the assumption of rigid diaphragms (0.15g versus 0.22g for the south wall of the firehouse). However, the opposite may be true when soil flexibility is considered. In this case, wall accelerations were computed to be larger with flexible diaphragms than with rigid diaphragms (0.40g versus 0.31g). Although the soil was identified as stiff, and the foundation consisted of spread footings, the consideration of foundation flexibility was essential for good correlation with the measured dynamic response of the firehouse.

TABLE 1. Roof Accelerations for Firehouse (g)

	Flexible Diaphragms			Rigid Diaphragms	
Element (1)	Measured (2)	Fixed base (3)	Flexible base (4)	Fixed base. (5)	Flexible base (6)
(*a*) East-west direction					
South wall	—	0.15	0.40	0.22	0.31
Central wall	0.41	0.06	0.34	0.22	0.31
North wall	—	0.04	0.31	0.22	0.31
South diaphragm	0.79	0.55	0.79	0.22	0.31
North diaphragm	—	0.17	0.39	0.22	0.31
(*b*) North-south direction					
East wall	—	0.20	0.30	0.22	0.24
West wall	—	0.08	0.20	0.22	0.24
South diaphragm	0.55	0.44	0.54	0.22	0.24
North diaphragm	—	0.43	0.53	0.22	0.24

TABLE 2. Roof Accelerations for Office Building (g)

	Flexible Diaphragms			Rigid Diaphragms	
Element (1)	Measured (2)	Fixed base (3)	Flexible base (4)	Fixed base (5)	Flexible base (6)
(*a*) East-west direction					
South wall	0.32	0.06	0.21	0.17	0.21
North wall	0.34	0.06	0.21	0.17	0.21
Diaphragm	0.53	0.44	0.55	0.17	0.21
(*b*) North-south direction					
South wall	—	0.07	0.16	0.21	0.26
North wall	—	0.07	0.16	0.21	0.26
Diaphragm	0.36	0.37	0.45	0.21	0.26

According to the model of the firehouse in the east-west direction, the most flexible wall [south wall, Fig. 7(a)] should accelerate slightly more than the adjacent parallel walls (0.40g versus 0.34g and 0.31g) when diaphragms and the soil are assumed to be flexible. Similar observations can be drawn in the north-south direction where the more flexible east wall should accelerate more than the west wall (0.30g versus 0.20g).

Flexible diaphragms should accelerate more than stiffer diaphragms. In the east-west direction, peak acceleration at the midspan of the south firehouse roof were measured to be the same 0.79g as computed, and was larger than the estimated acceleration for the shorter north diaphragm span (0.39g). These values should decrease to 0.31g with a rigid diaphragm according to the model.

The computed response of the office building (Table 2) infers similar, but less pronounced trends. Wall accelerations were computed to be much less with flexible diaphragms than with rigid diaphragms when a fixed-base condition was considered (0.06g versus 0.17g in the east-west direction), but were the same (0.21g in the east-west direction) for both rigid- and flexible-diaphragm assumptions when soil flexibility was considered. Roof accelerations were again much larger when diaphragms were considered to be flexible (0.55g versus 0.21g in the east-west direction).

Maximum Lateral Displacements

Computed peak deflections for flexible- and rigid-diaphragm systems are shown in Table 3 for the firehouse and in Table 4 for the office building.

TABLE 3. Roof Displacements for Firehouse (mm)

Element (1)	Flexible Diaphragms Measured (2)	Fixed base (3)	Flexible base (4)	Rigid Diaphragms Fixed base (5)	Flexible base (6)
(*a*) East-west direction					
South wall	—	5.1	21.8	2.0	14.5
Central wall	—	2.0	18.3	2.0	14.5
North wall	—	0.8	17.0	2.0	14.5
South diaphragm	—	22.1	40.6	2.0	14.5
North diaphragm	—	4.3	21.1	2.0	14.5
(*b*) North-south direction					
East wall	—	3.0	6.6	1.5	2.4
West wall	—	1.3	4.6	1.5	2.4
South diaphragm	—	12.4	16.3	1.5	2.4
North diaphragm	—	12.2	16.0	1.5	2.4

TABLE 4. Roof Displacements for Office Building (mm)

Element (1)	Flexible Diaphragms Measured (2)	Fixed base (3)	Flexible base (4)	Rigid Diaphragms Fixed base (5)	Flexible base (6)
(*a*) East-west direction					
South wall	—	1.3	5.1	1.5	4.6
North wall	—	1.3	5.1	1.5	4.6
Diaphragm	—	20.6	26.7	1.5	4.6
(*b*) North-south direction					
South wall	—	1.8	5.1	2.5	5.3
North wall	—	1.8	5.1	2.5	5.3
Diaphragm	—	12.7	16.3	2.5	5.3

According to the model, the most flexible wall of the firehouse (south wall) should deflect more than the adjacent parallel walls when a flexible diaphragm is assumed (2.18 cm versus 1.83 cm, and 1.70 cm for a flexible soil). With a rigid diaphragm, deflections of all walls should be equal by definition, and generally less than those for the flexible-diaphragm system, particularly when the soil is considered to be flexible. Diaphragm deflections should be much larger for flexible than for rigid systems; however, the ratio of diaphragm deflections for flexible and rigid systems should reduce as soil flexibility is increased. Similar, but less pronounced trends can be seen with computed displacements of the office building.

Although the analytical models only considered the in-plane masonry shear walls, diaphragm deflections will impose large deflections on the transverse walls. This action can result in significant tensile stresses normal to the bed joints, and may cause horizontal cracking at the brick-mortar interface. No damage of the firehouse was observed in this context because the imposed out-of-plane wall deformations were small. Finite-element idealizations of transverse bending in the south and east walls were done to identify critical tensile stresses normal to the head joints at times of peak displacement. Peak tensile stresses normal to the head joints were determined to be only slightly larger than the allowable working stresses prescribed by the 1994 UBC (Tena-Colunga and Abams 1992b).

Torsional Effects

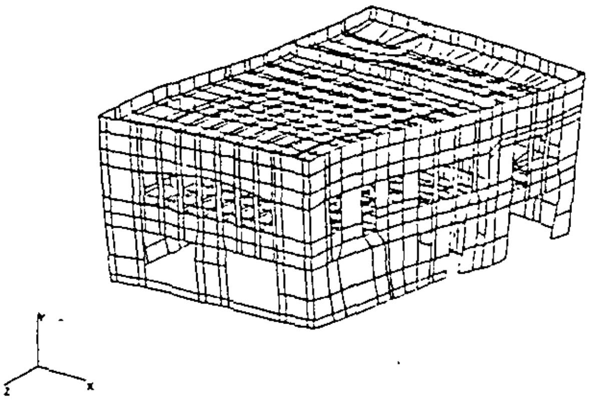

FIG. 9. Torsional Mode Shape for Firehouse

The influence of diaphragm flexibility on torsional effects was examined using a three-dimensional finite-element model to study the vibrational characteristics of the firehouse. The finite-element code *ABAQUS* (1989) was used for the analyses with three-dimensional thick shell elements for the masonry walls and diaphragms. The second global mode shape (Fig. 9) defined a torsional mode characterized by significant rotational components in the x-z plane. However, the influence of this mode was diminished as the flexibility of the diaphragm was increased. The associated modal masses in the north-south (the z direction) and the east-west (the x direction) were relatively low (0.7% and 7.5% of the total modal mass, respectively).

Another set of analyses was done considering rigid diaphragms. A matrix method was used to compute the torsional eccentricity of the structure (Damy-Ríos and Alcocer 1987). The torsional eccentricities as fractions of the maximum plan dimension were, for the east-west direction: $e_x = 0.20L_1$ (first floor), $e_x = 0.23L_1$ (roof level), and $L_1 = 12.2$ m; and for the north-south direction: $e_z = 0.31L_2$ (first floor), $e_z = 0.14L_2$ (roof level), and $L_2 = 19.0$ m. The first two mode shapes and associated modal masses were computed using standard methods of structural dynamics. For the first mode, the associated modal masses were 52.7 and 29.9% of the total modal mass in the east-west and north-south directions, respectively, whereas for the second mode the associated modal masses were 32.9 and 47.7%. These analyses confirmed the high torsional coupling that the firehouse should have experienced if the diaphragms were considered to be rigid.

A computational study of the Oakland hotel structure also revealed the importance of a flexible diaphragm in reducing torsional effects. As for the firehouse, a three-dimensional finite-element model was used to define the vibrational characteristics of the hotel considering that the diaphragms were flexible or rigid. The first four modes computed for both diaphragm assumptions are summarized in Table 5. Modal masses are presented as a fraction of the total modal mass acting in the identified direction. Torsional coupling is much higher for the rigid-diaphragm assumption. For the first mode, the associated modal mass in the north-south direction decreased from 75.8% of the global mass to only 56.8% when the diaphragms were considered to be rigid. In addition, the associated modal mass in the orthogonal direction increased from 2.0 to 16.3% for this mode, which illustrates torsional

TABLE 5. Mode Shapes for Hotel

Mode (1)	FLEXIBLE DIAPHRAGMS Period (s) (2)	Associated Modal Mass x (3)	y (4)	z (5)	RIGID DIAPHRAGMS Period (s) (6)	Associated Modal Mass x (7)	y (8)	z (9)
1	1.11	0.02	0.00	0.76	0.97	0.16	0.00	0.57
2	0.80	0.83	0.00	0.02	0.43	0.58	0.01	0.25
3	0.49	0.01	0.00	0.07	0.27	0.14	0.00	0.06
4	0.40	0.00	0.04	0.00	0.20	0.02	0.56	0.03

coupling with the rigid-diaphragm idealization. This tendency was even more pronounced for the second mode.

Natural Period

For estimation of the fundamental period of vibration, most seismic codes do not differentiate between building systems with flexible diaphragms from those with rigid diaphragms.

Method A of the 1994 UBC code (Section 1628.2.2) is an approximate method intended for all buildings. The period estimate is given by Eq. (28-3) of the code, which for shear-wall systems (masonry or reinforced concrete) can be rewritten as

$$T = \frac{0.0743}{\sqrt{\sum A_e \left[0.2 + \left(\frac{D_e}{h_n}\right)^2\right]}} h_n^{(3/4)} \tag{1}$$

where A_e = minimum cross-sectional shear area in any horizontal plane of a shear wall in the first story (in m^2); D_e = length of a wall in the direction parallel to the applied forces in the first story (in m); and h_n = height of the building (in m). The D_e/h_n ratio shall not exceed 0.9. This equation is based on experimental data from structural systems with rigid diaphragms, and may therefore be inaccurate for flexible-diaphragm systems. Method B of the 1994 UBC is based on the Rayleigh quotient and the fundamental period is determined using the structural properties and deformation characteristics of the resisting elements. If a suitable model is used for the analysis of structures with flexible diaphragms, the natural period estimate should be closely predicted using this method.

The National Earthquake Hazards Reduction Program (NEHRP) ("NEHRP" 1994) provisions state that the period of a building structure shall be based on established methods of mechanics and the properties of structural systems in the direction of analysis. The base of the building is assumed to be fixed. The natural period as calculated shall not exceed the approximate period of the structure as estimated with the following:

$$T \leq C_a T_a \tag{2}$$

where C_a = a coefficient for an upper limit on calculated period [Table 2.3.3, "NEHRP" (1994a)]; and T_a = approximate fundamental period of the building, which for a shear-wall building is computed as

$$T_a = 0.049 h_n^{(3/4)} \tag{3}$$

where h_n = height (in m) between the base and the highest level of the building. Eq. (3) is intended to give a conservative shorter period for use in estimating an equivalent base shear ("NEHRP" 1994b). Good correlations cannot necessarily be expected for structures with flexible diaphragms.

The 1994 NEHRP provisions also include soil-structure-interaction effects. The effective fundamental period of a building structure can be determined using Eq. 2.5.2.1.1-1 of the provisions, which is based on the ATC 3-06 document ("Tentative" 1978) and is reproduced as follows:

$$\bar{T} = T \sqrt{\left(1 + \frac{\bar{k}}{K_y}\right)\left(1 + \frac{K_y \bar{h}^2}{K_\theta}\right)} \tag{4}$$

where T = period as determined with (2); $\bar{k}$ = stiffness of a fixed-base building; $\bar{h}$ = effective height of the building or 0.7 times the total height; K_y = lateral foundation stiffness; and K_θ = rocking stiffness of the foundation. Because (4) includes the stiffness of the soil and structure, a period estimate is reasonably accurate if a suitable model is used for a flexible-diaphragm system.

TABLE 6. Fundamental Periods and Code Estimates (s)

Building (1)	Period (2)	UBCA (3)	UBCB (4)	NEHRP fixed base (5)	NEHRP with soil (6
Firehouse, NS	0.325	0.154	0.321	0.234	0.3
Firehouse, EW	0.453	0.138	0.453	0.234	0.40
Office building, NS	0.367	0.210	0.350	0.216	0.370
Office building, EW	0.400	0.300	0.370	0.216	0.420
Hotel, NS	1.113	0.488	1.102	0.594	1.125
Hotel, EW	0.804	0.519	0.793	0.594	0.823

Periods of vibration per the UBC and NEHRP provisions are compared in Table 6 with the measured periods of the Gilroy firehouse and the Palo Alto office building. Periods were read from Fourier amplitude spectra computed from measured acceleration records. The computed fundamental period of the Oakland hotel from three-dimensional finite-element analyses was used in lieu of measured data. The UBC Method A and NEHRP fixed-base periods consistently provided shorter periods than were inferred from measurements or were computed. Furthermore, these period estimates differ substantially between themselves. Because the design spectra for both the UBC and the NEHRP provisions result in larger design forces as the natural period gets shorter (based on a stiff soil), the major effect of underestimating the natural period of a structure is that the base shear will be overestimated. Hence, the use of these approximate periods will be conservative. However, for softer soils, higher accelerations are expected for structures with periods in the medium range, and the underestimating period will not necessarily be conservative.

Period estimates based on the more rigorous code methods (UBC Method B and NEHRP with soil-structure interaction) were in close agreement with measured and finite-elen method values. For these estimates, the discrete MDOF mo (Figs. 7 and 8) were used to determine the structural properties of the building systems.

SUMMARY AND CONCLUSIONS

The influence of floor flexibility was examined for building structures with either flexible or rigid diaphragms. The dynamic characteristics of three existing building systems with flexible diaphragms, and counterpart hypothetical systems with rigid diaphragms, were contrasted to help illustrate the significance of flexible-diaphragm behavior on seismic response.

Analytical studies suggested that, as diaphragm flexibility increases, diaphragm and shear-wall accelerations can increase in some cases. Flexible in-plane shear walls can vibrate at higher accelerations than stiffer walls in a flexible-diaphragm system. Design criteria based on rigid-diaphragm behavior is not necessarily conservative for flexible-diaphragm systems.

Torsional effects can be reduced considerably as the flexibility of the diaphragm is increased.

The fundamental period of building systems with flexible diaphragms is consistently longer than values estimated with simplified methods given by current seismic codes.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

A_e = minimum cross-sectional shear area of shear wall in any horizontal plane at first story;
C_a = coefficient for an upper limit on period;
D_e = length of wall in the direction parallel with shear forces;
e_x, e_z = torsional eccentricies in the x and z directions;
$\bar{h}$ = effective height of building, 0.7 times full height;
h_n = height of building;
K_θ = rocking stiffness of foundation;
K_y = lateral stiffness of foundation;
$\tilde{k}$ = stiffness of fixed-base building;
L = length of building in direction being analyzed;
L_1, L_2 = length of building parallel to torsional eccentricities;
T = fundamental period of building; and
T_a = approximate fundamental period per Eq. (3).

FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

XXVI CURSO INTERNACIONAL DE INGENIERIA SÍSMICA

MODULO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

NORMAS TÉCNICAS COMPLEMENTARIAS PARA DISEÑO Y CONSTRUCCIÓN DE ESTRUCTURAS DE MAMPOSTERÍA

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PALACIO DE MINERIA
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NORMAS TECNICAS COMPLEMENTARIAS PARA DISEÑO Y CONSTRUCCION DE ESTRUCTURAS DE MAMPOSTERIA

INDICE

NORMAS TECNICAS COMPLEMENTARIAS PARA DISEÑO Y CONSTRUCCION DE ESTRUCTURAS DE MAMPOSTERIA

OSCAR ESPINOSA VILLARREAL, Jefe del Departamento del Distrito Federal, con fundamento en los artículos 122, Fracción VI, de la Constitución Política de los Estados Unidos Mexicanos, en relación con el quinto Transitorio del Decreto que reforma a la propia Constitución, publicado en el *Diario Oficial de la Federación* el 25 de octubre de 1993; 67, Fracción XXII, del Estatuto de Gobierno del Distrito Federal, 4 12 y 24 de la Ley Orgánica de la Administración Pública del Distrito Federal; 3o., Fracción XV y Séptimo Transitorio del Reglamento de Construcciones para el Distrito Federal, y el Acuerdo por el que deberán expedirse las Normas Técnicas Complementarias del Reglamento de Construcciones para el Distrito Federal, publicado en la *Gaceta Oficial del Departamento del Distrito Federal* el 7 de noviembre de 1994, he tenido a bien expedir las siguientes:

NORMAS TECNICAS COMPLEMENTARIAS PARA DISEÑO Y CONSTRUCCION DE ESTRUCTURAS DE MAMPOSTERIA

Normas Técnicas Complementarias para Diseño y Construcción de Estructuras de Mampostería

NOTACION

A_s	área de acero de refuerzo colocada en el extremo de un muro
A_{sh}	área total de refuerzo horizontal en el muro
A_{sv}	área total de refuerzo vertical en el muro
A_T	área bruta de la sección transversal del muro
B	coeficiente para el cálculo de la resistencia ante carga vertical de muros rigidizados por elementos transversales
b	longitud de apoyo de una losa soportada por el muro
c_p	coeficiente de variación de la resistencia en compresión de las piezas
c_m	coeficiente de variación de la resistencia en compresión de la mampostería
c_v	coeficiente de variación de la resistencia en cortante de la mampostería
d	distancia entre el centroide del acero de tensión y el extremo opuesto del muro
d_c	menor dimensión de la sección del castillo o cadena que confina al muro.
d'	distancia entre los centroides del acero colocado en ambos extremos de un muro
E	módulo de elasticidad de la mampostería para esfuerzos de compresión normales a las juntas
F_E	factor de reducción por efectos de excentricidad y esbeltez
F_R	factor de reducción de resistencia
f'_c	resistencia especificada del concreto en compresión
$\bar{f}_m$	media de la resistencia en compresión de la mampostería, referida al área bruta
f^*_m	resistencia de diseño en compresión de la mampostería, referida al área bruta
$\bar{f}_p$	media de la resistencia en compresión de las piezas, referida al área bruta
f^*_p	resistencia de diseño en compresión de las piezas, referida al área bruta
f_y	esfuerzo de fluencia especificado del acero de refuerzo
G	módulo de cortante de la mampostería
H	altura no restringida del muro
H'	altura efectiva del muro
L	longitud efectiva del muro
L'	separación entre elementos que rigidizan transversalmente al muro
M_r	momento flexionante, aplicado en el plano, que resiste el muro en flexocompresión
M_o	momento flexionante, aplicado en el plano, que resiste el muro en flexión pura
P	carga axial total que obra sobre el muro, sin multiplicar por el factor de carga
P_u	carga axial total que obra sobre el muro multiplicada por el factor de carga
P_R	resistencia de diseño del muro a carga vertical
p_h	cuantía de refuerzo horizontal en el muro
p_v	cuantía de refuerzo vertical en el muro
Q	factor de comportamiento sísmico
s	separación del acero de refuerzo
t	espesor del muro
V_R	fuerza cortante resistente
v^*	esfuerzo cortante de diseño, sobre área bruta
$\bar{v}$	media de los esfuerzos cortantes resistentes de muretes, sobre área bruta

1. CONSIDERACIONES GENERALES

1.1 Alcance

Los capítulos 2 a 5 de estas disposiciones se aplican al diseño y construcción de muros constituídos por piezas prismáticas de piedra artificial, macizas o huecas, unidas por un mortero aglutinante. Incluyen muros reforzados con armados interiores, castillos, cadenas o contrafuertes.

El capítulo 6 se aplica al diseño y construcción de elementos de mampostería de piedras naturales.

2. MATERIALES PARA MAMPOSTERIA

2.1 Piezas

2.1.1 Tipos de piezas

Las piezas usadas en los elementos estructurales de mampostería deberán cumplir los requisitos generales de calidad especificados por la Dirección General de Normas de la Secretaría de Comercio y Fomento Industrial para cada material. En particular deberán aplicarse las siguientes normas.

C6 Ladrillos y bloques cerámicos de barro, arcilla o similares

C10 Bloques, ladrillos o tabiques y tabicones de concreto

En el capítulo de diseño sísmico del Reglamento se fijan distintos factores de comportamiento sísmico, Q, en función del tipo de pieza que compone un muro y de su refuerzo

Para fines de aplicación del capítulo mencionado se considerarán como piezas macizas aquellas que tienen en su sección transversal más desfavorable un área neta de por los menos 75 por ciento del área total, y cuyas paredes no tienen espesores menores de 2 cm.

Las piezas huecas a que hace referencia el capítulo de diseño sísmico son las que tienen en su sección transversal más desfavorable un área neta de por lo menos 45 por ciento del área bruta; además el espesor de sus paredes exteriores no es menor que 1.5 cm.

2.1.2 Resistencia en compresión

La resistencia en compresión se determinará para cada tipo de piezas de acuerdo con el ensaye especificado en la norma NOM C 36.

Para diseño se empleará un valor de la resistencia, f_p^*, medida sobre el área bruta, que se determinará como el que es alcanzado por lo menos por el 98% de las piezas producidas.

Cuando se tenga evidencia de que el valor mínimo garantizado por el fabricante cumple con la definición anterior, podrá tomarse éste como resistencia de diseño.

Cuando no se cumpla lo anterior, la resistencia de diseño se determinará con base en la información estadística existente sobre el producto en cuestión o a partir de muestreos de la producción de la pieza en cuestión. En este último caso se obtendrán al menos tres muestras de diez piezas cada una, de lotes diferentes de la producción. Las 30 piezas así obtenidas se ensayarán con el procedimiento especificado en la norma C36 y la resistencia de diseño se calculará como.

$$f_p^* = \frac{\bar{f}_p}{1 + 2.5c_p}$$

donde:

$\bar{f}_p$ es el promedio de las resistencias en compresión de las piezas ensayadas

c_p es el coeficiente de variación de la resistencia de las piezas ensayadas, pero su valor no se tomará menor que 0.20 para piezas provenientes de plantas mecanizadas con control de calidad de la resistencia, que 0.30 para piezas de fabricación mecanizada, pero sin control de calidad de resistencia, y que 0.35 para piezas de producción artesanal.

2.2 Morteros

Los morteros que se empleen en elementos estructurales de mampostería deberán cumplir con los requisitos siguientes:

a) Su resistencia en compresión será por lo menos de 40 kg/cm².

b) La relación volumétrica entre la arena y la suma de cementantes se encontrará entre 2.25 y 3.

c) La resistencia se determinará según lo especificado en la norma NOM C 61.

d) Se empleará la mínima cantidad de agua que dé como resultado un mortero fácilmente trabajable

La tabla siguiente muestra las características de algunos proporcionamientos recomendados.

PROPORCIONAMIENTOS, EN VOLUMEN, RECOMENDADOS PARA MORTERO EN ELEMENTOS ESTRUCTURALES

Tipo de mortero	Partes de cemento	Partes de cemento de albañilería	Partes de cal	Partes de arena*	Valor típico de la reistencia nominal en compresion en kg/cm²
I	1	-	0 a $^1/_4$	No menos de 2.25 ni más de 3 veces la suma de cementantes en volumen	125
	1	0 a $^1/_2$	-		
II	1	-	$^1/_4$ a $^1/_2$		75
	1	$^1/_2$ a 1	-		
III	1	-	$^1/_2$ a $1^1/_4$		40

* El volumen de arena se medirá en estado suelto.

2.3 Acero de refuerzo

El refuerzo que se emplee en castillos, dalas y/o elementos colocados en el interior del muro, estará constituido por barras corrugadas que cumplan las especificaciones NOM B6 y NOM B294, por malla de acero que cumpla con la especificación B290 o por alambres corrugados laminados en frío que cumplan con la norma NOM B72, o por armaduras soldadas por resistencia eléctrica de alambre de acero para castillos y dalas que cumplan con la norma NOM B-456. Se admitirá el uso de barras lisas únicamente en estribos, en mallas electrosoldadas o en conectores. Se podrán utilizar otros tipos de acero siempre y cuando se demuestre a satisfacción del Departamento su eficiencia como refuerzo estructural.

Como esfuerzo de diseño, f_y, se considerará el de fluencia garantizado por el fabricante. La verificación de calidad del acero se hará de acuerdo con la norma correspondiente de la Dirección General de Normas.

2.- Mampostería

2 - Resistencia a compresión.

La resistencia de diseño en compresión de la mampostería, f^*_m, sobre área bruta, se determinará con alguno de los procedimientos siguientes:

a) *Ensayes de pilas construidas con las piezas y morteros que se emplearán en la obra.* Las pilas estarán formadas por lo menos con 3 piezas sobrepuestas. La relación altura espesor de la pila estará comprendida entre 2 y 5; las pilas se ensayarán a la edad de 28 días. Para el almacenamiento de los especímenes, su cabeceado y el procedimiento de ensaye se seguirán, en lo que sean aplicables, las normas que rigen para el ensaye a compresión de cilindros de concreto (NOM C83).

El esfuerzo medio obtenido, calculado sobre el área bruta, se corregirá multiplicándolo por los factores de la tabla siguiente:

FACTORES CORRECTIVOS PARA LAS RESISTENCIAS DE PILAS CON DIFERENTES RELACIONES DE ESBELTEZ

Relación de esbeltez de la pila	2	3	4	5
Factor correctivo	0.75	0.90	1.00	1.05

Para esbelteces intermedias se interpolará linealmente. La resistencia de diseño se calculará como

$$f^*_m = \frac{\bar{f}_m}{1 + 2.5c_m}$$

en que

$\bar{f}_m$ es el promedio de la resistencia de las pilas ensayadas, corregida por esbeltez

c_m el coeficiente de variación de la resistencia de las pilas ensayadas, que en ningún caso se tomará inferior a 0.15

La determinación se hará en un minimo de 9 pilas construidas con piezas provenientes de por lo menos 3 lotes diferentes del mismo producto

b) *A partir de la resistencia de diseño de las piezas el mortero*

1. Para bloques y tabique de concreto con relación altura a espesor no menor que un medio, y con $f^*_p \leq 200$ kg/cm², la resistencia de diseño a compresión será la que indica la tabla siguiente, si se comprueba que las piezas y el mortero cumplen con los requisitos de calidad especificados en 2.1 y 2.2, respectivamente.

RESISTENCIA DE DISEÑO A COMPRESION DE LA MAMPOSTERIA DE PIEZAS DE CONCRETO (f^*_m SOBRE AREA BRUTA)

	f^*_m, en kg/cm²		
f^*_p, en kg/cm²	Mortero I	Mortero II	Mortero III
25	15	10	10
50	25	20	20
75	40	35	30
100	50	45	40
150	75	60	60
200	100	90	80

Para valores intermedios se interpolará linealmente.

2. Para piezas de barro y otros materiales, excepto concreto, con relación altura a espesor no menor que un medio, la resistencia de diseño a compresión será lo que se obtiene de la tabla siguiente para los morteros recomendados

RESISTENCIA DE DISEÑO A COMPRESION DE LA MAMPOSTERIA DE PIEZAS DE BARRO (f^*_m, SOBRE AREA BRUTA)

	f^*_m, en kg/cm²		
f^*_p, en kg/cm²	Mortero I	Mortero II	Mortero III
25	10	10	10
50	20	20	20
75	30	30	25
100	40	40	30
150	60	60	40
200	80	70	50
300	120	90	70
400	140	110	90
500	160	130	110

Para valores intermedios se interpolará linealmente.

c) *Valores indicativos.* Si no se realizan determinaciones experimentales podrán emplearse los valores de f^*_m que, para distintos tipos de piezas y morteros, se presentan en la tabla siguiente:

RESISTENCIA DE DISEÑO A COMPRESION DE LA MAMPOSTERIA, f^*_m, PARA ALGUNOS TIPOS DE PIEZA, SOBRE AREA BRUTA[1]

Tipo de pieza	Valores de f^*_m, en kg/cm²		
	Mortero I	Mortero II	Mortero III
Tabique de barro recocido	15	15	15
Bloque de concreto tipo A (pesado)	20	15	15
Tabique de concreto[2] (f^*_p>80 kg/cm²)	20	15	15
Tabique con huecos verticales[1] (f^*_p>120 kg/cm²)	40	40	30

[1] La relación área neta-bruta no será menor de 0.45.

[2] Fabricado con arena sílica y peso volumétrico no menor de 1500kg/m³.

d) *Resistencia en compresión de mampostería con refuerzo interior.* Para mampostería con refuerzo interior que cumpla con los requisitos especificados en 3.4, se tomará para f^*_m el valor que corresponde a mampostería sin refuerzo, incrementado en 25%, pero no en más de 7 kg/cm².

e) *Resistencia en compresión de muros confinados.* Para muros reforzados con dalas y castillos que cumplan los requisitos de 3.3, el esfuerzo resistente en compresión, f^*_m, calculado para la mampostería sin refuerzo podrá incrementarse en 4 kg/cm².

2 . . *Esfuerzo cortante resistente de diseño*

La resistencia a fuerza cortante de muros de mampostería según se calcula en la sección 4.3, se basa en el esfuerzo cortante resistente de diseño, v*, el cual se tomará de la tabla siguiente:

ESFUERZO CORTANTE RESISTENTE DE DISEÑO PARA ALGUNOS TIPOS DE MAMPOSTERIA, SOBRE AREA BRUTA

Pieza	Tipo de mortero	v*[1] en kg/cm²
Tabique de barro recocido	I	3.5
	II y III	3
Tabique de concreto (f^*_p> 80 kg/cm²)	I	3
	II y III	2
Tabique hueco de barro[2]	I	3
	II y III	2
Bloque de concreto tipo A (pesado)	I	3.5
	II y III	2.5

[1] Las piezas huecas deberán cumplir con los requisitos fijados en 2.1. Cuando el valor de la tabla sea mayor que $0.8\sqrt{f^*_m}$ se tomará este último valor como v*

[2] Tabique de barro con perforaciones verticales con relación de áreas neta a bruta no menor de 0.45

Para materiales no cubiertos en la tabla anterior el esfuerzo cortante resistente se determinará mediante ensayes con procedimientos aprobados por el Departamento.

Será aceptable la determinación del esfuerzo cortante resistente a partir del ensaye de muretes con una longitud de al menos una vez y media la máxima dimensión de la pieza y con el número de hiladas necesario para que la altura sea aproximadamente igual a la longitud. Los muretes se ensayarán sometiéndolos a una carga de compresión a lo largo de su diagonal y el esfuerzo cortante medio se determinará dividiendo la carga máxima entre el área bruta del murete medida sobre la misma diagonal.

La determinación se hará sobre un mínimo de 9 muretes construidos con piezas provenientes de por lo menos tres lotes diferentes.

Para diseño se utilizará un esfuerzo resistente igual a

$$v^* = \frac{\bar{v}}{1 - 2.5c_v}$$

en que

$\bar{v}$ es el promedio de los esfuerzos resistentes de los muretes ensayados

c_v es el coeficiente de variación de los esfuerzos resistentes de los muretes ensayados que no se tomará menor que 0.20

Para muros que dispongan de algún sistema de refuerzo cuya contribución a la resistencia se quiera evaluar o que tengan características que no pueden representarse en el tamaño del murete, las pruebas de compresión diagonal antes descritas deberán realizarse en muros de al menos 2 × 2 m.

2 · 3 Resistencia al aplastamiento

Cuando una carga concentrada se transmite directamente a la mampostería, el esfuerzo de contacto no excederá de 0.6 f^*_m. El esfuerzo actuante se calculará con las cargas de diseño obtenidas aplicando los factores correspondientes a la combinación de acciones de que se trate según el artículo 194 del Reglamento.

2 · 4 Resistencia a tensión

Se considerará que es nula la resistencia de la mampostería a esfuerzos de tensión perpendiculares a las juntas. Cuando se requiera esta resistencia deberá proporcionarse el refuerzo necesario.

2 · 5 Módulo de elasticidad

El módulo de elasticidad de la mampostería, E, podrá determinarse experimentalmente o calcularse en forma aproximada como sigue:

Para mampostería de tabiques y bloques de concreto.

$E = 800 f^*_m$ para cargas de corta duración

$E = 350 f^*_m$ para cargas sostenidas

Para mampostería de tabique de barro y otras piezas, excepto las de concreto

$E = 600 f^*_m$ para cargas de corta duración

$E = 350 f^*_m$ para cargas sostenidas

2 · 6 Módulo de cortante

El módulo de cortante de la mampostería se tomará como

$$G = 0.3E$$

3. SISTEMAS ESTRUCTURALES A BASE DE MUROS DE MAMPOSTERIA

3.1 Tipos de muros

Los muros que tengan una función estructural en la construcción quedarán incluidos en una de las modalidades descritas en los casos siguientes.

3.2 Muros diafragma

Estos son los que se encuentran rodeados por las vigas y columnas de un marco estructural al que proporcionan rigidez ante cargas laterales.

La unión entre el marco y el muro diafragma deberá evitar la posibilidad de volteo del muro perpendicularmente a su plano y las columnas del marco deberán ser capaces de resistir, cada una, en una longitud igual a una cuarta parte de su altura medida a partir del paño de la viga, una fuerza cortante igual a la mitad de la carga lateral que actúa sobre el tablero.

3.3 Muros confinados

Estos son los que están reforzados con castillos y dalas que cumplen con los requisitos siguientes:

Las dalas y castillos tendrán como dimensión mínima el espesor del muro. El concreto tendrá una resistencia a compresión, f'_c no menor de 150 kg/cm², y el refuerzo longitudinal estará formado por lo menos de tres barras, cuya área no será inferior a $0.2\, f'_c/f_y$ multiplicado por el cuadrado del espesor del muro, t^2, y estará anclado en los elementos que limitan al muro de manera que pueda desarrollar su esfuerzo de fluencia.

El área del refuerzo transversal no será inferior a $\frac{1000s}{f_y d_c}$ siendo s la separación de estribos y d_c la menor dimensión de la sección del castillo o dala. La separación de los estribos no excederá de 1.5 d_c ni de 20 cm.

Existirán castillos por lo menos en los extremos de los muros y en puntos intermedios del muro a una separación no mayor que vez y media su altura, ni 4 m.

Existirá una dala en todo extremo horizontal de muro, a menos que este último esté ligado a un elemento de concreto reforzado de al menos 15 cm de peralte. Además existirán dalas en el interior del muro a una separación no mayor de 3 m.

Existirán elementos de refuerzo con las mismas características que las dalas y castillos en el perímetro de todo hueco cuya dimensión exceda de la cuarta parte de la longitud del muro en la misma dirección.

La relación altura a espesor del muro no excederá de 30.

Podrá incrementarse la resistencia a fuerza cortante de muros confinados, de acuerdo con lo establecido en 4.3.2, cuando se coloque refuerzo horizontal en las juntas con las cuantías mínimas especificadas en dicha sección y que cumpla con los requisitos de separación máxima y de detallado especificados para muros reforzados interiormente en la sección 3.4. Dicho refuerzo horizontal deberá estar anclado a los castillos extremos e nteriores

3.4 Muros reforzados interiormente

Estos son muros reforzados con malla o barras corrugadas de acero, horizontales y verticales, colocadas en los huecos de las piezas, en ductos o en las juntas. Para que un muro pueda considerarse como reforzado deberán cumplirse los siguientes requisitos mínimos.

La suma de la cuantía de refuerzo horizontal, p_h, y vertical, p_v, no será menor que 0.002 y ninguna de las dos cuantías será menor que 0.0007. La cuantía de refuerzo horizontal se calcula como $p_h = A_{sh}/st$, donde A_{sh} es el refuerzo horizontal que se colocará en el espesor t del muro a una separación s. $p_v = A_{sv}/tL$, en que A_{sv} es el área total de refuerzo que se colocará verticalmente en la longitud L del muro. Cuando se emplee acero de refuerzo de fluencia especificado mayor de 4200 kg/cm², las cuantías de refuerzo mencionadas en este párrafo podrán reducirse multiplicándolas por $4200/f_y$.

Todo espacio que contenga una barra de refuerzo vertical deberá tener una distancia libre mínima entre el refuerzo y las paredes de la pieza igual a la mitad del diámetro de la barra y deberá ser llenado a todo lo largo con mortero o concreto. La distancia libre mínima entre una barra de refuerzo horizontal y el exterior del muro será de 1.5 cm o una vez el diámetro de la barra, la que resulte mayor. El refuerzo horizontal deberá estar embebido en toda su longitud en mortero o concreto.

Para el colado de los huecos donde se aloje el refuerzo vertical podrá emplearse el mismo mortero que se usa para pegar las piezas o un concreto de alto revenimiento, con agregado máximo de 1 cm y resistencia a compresión no menor de 125 kg/cm². El hueco de las piezas tendrá una dimensión mínima mayor de 5 cm y un área no menor de 30 cm²

Deberá colocarse por lo menos una barra No. 3 de grado 42, o refuerzo de otras características con resistencia a tensión equivalente, en dos huecos consecutivos en todo extremo de muros, en las intersecciones entre muros o a cada 3 m. El refuerzo vertical en el interior del muro tendrá una separación no mayor de 6 veces el espesor del mismo ni mayor de 80 cm

Cuando los muros transversales lleguen a tope, sin traslape de piezas, será necesario unirlos mediante dispositivos que aseguren la continuidad de la estructura.

El refuerzo horizontal debe ser continuo y sin traslape en la longitud del muro y anclado en sus extremos. Se deberán cumplir los mismos requisitos de anclaje que para concreto reforzado. Deberá haber refuerzo consistente en una barra No 4 de grado 42, o con resistencia a tensión equivalente, alrededor de toda abertura cuya dimensión exceda de 60 cm en cualquier dirección.

La relación altura/espesor de estos muros no será superior a 30

Deberá haber una supervisión continua en la obra que asegure que el refuerzo esté colocado de acuerdo con lo indicado en planos y que los huecos en que se aloja el refuerzo sean colados completamente.

3.5 Muros no reforzados

Se considerarán como muros no reforzados aquellos que no tengan el refuerzo necesario para ser incluídos en alguna de las tres categorías anteriores.

3.6 Otras modalidades de refuerzo y construcción de muros

Cualquier otro tipo de refuerzo o de modalidad constructiva a base de mampostería deberá ser avalado por evidencia experimental y analítica que demuestre, a satisfacción del Departamento, que cumple con los requisitos de seguridad estructural establecidos por el Reglamento y por estas Normas

4. PROCEDIMIENTO DE DISEÑO

4.1 Análisis

4.1.1 Criterio General

La determinación de las fuerzas internas en los muros se hará en general por medio de un análisis elástico En la determinación de las propiedades elásticas de los muros deberá considerarse que la mampostería no resiste tensiones en dirección normal a las juntas y emplear por tanto las propiedades de las secciones agrietadas y transformadas cuando dichas tensiones aparezcan.

4.1.2 Análisis por cargas verticales

Para el análisis por cargas verticales se tomará en cuenta que en las juntas de los muros y los elementos de piso ocurren rotaciones locales debidas al aplastamiento del mortero. Por tanto, para muros que soportan losas de concreto, la junta tiene suficiente capacidad de rotación para que pueda considerarse que, para efectos de la distribución de momentos en el nudo la rigidez de los muros es nula. Para el diseño sólo se tomarán en cuenta los momentos debidos a los efectos siguientes:

a) Los momentos que deben ser resistidos por condiciones de estática y que no pueden ser redistribuídos por la rotación del nudo, como son los momentos debidos a un voladizo que se empotre en el muro y los debidos a empujes, de viento o sismo, normales al plano del muro.

b) Los momentos debidos a la excentricidad con que se transmite la carga de la losa del piso inmediatamente superior en muros extremos; tal excentricidad se tomará igual a

$$e_c = \frac{t}{2} - \frac{b}{3}$$

en que t es el espesor del muro y b el de la porción de éste en que se apoya la losa soportada por éste.

Será admisible determinar únicamente las cargas verticales que actúan sobre cada muro mediante una bajada de cargas por áreas tributarias y tomar en cuenta los efectos de excentricidades y esbeltez mediante los valores aproximados del factor de reducción, F_E, recomendados en el caso I del inciso 4.2.2, cuando se cumplan las condiciones siguientes:

a) Las deformaciones de los extremos superior e inferior del muro en la dirección normal a su plano están restringidas por el sistema de piso o por otros elementos.

b) No hay excentricidad importante en la carga axial aplicada ni fuerzas significativas que actúan en dirección normal al plano del muro.

c) La relación altura espesor del muro no excede de 20.

4.1.[illegible] Análisis por cargas laterales

El análisis para la determinación de los efectos de las cargas laterales debidas a sismo se hará con base en las rigideces relativas de los distintos muros. Estas se determinarán tomando en cuenta las deformaciones de cortante y de flexión. Para estas últimas se considerará la sección transversal agrietada del muro cuando la relación de carga vertical a momento flexionante es tal que se presentan tensiones verticales. Se tomará en cuenta la restricción que impone a la rotación de los muros la rigidez de los sistemas de piso y techo y la de los dinteles.

Será admisible considerar que la fuerza cortante que toma cada muro es proporcional a su área transversal, ignorar los efectos de torsión y de momento de volteo, y emplear el método simplificado de diseño sísmico especificado en la sección 7 de las Normas Técnicas Complementarias de Diseño Sísmico, cuando se cumplan los requisitos especificados en la sección 2 de las normas citadas y que son los siguientes:

I. En todos los niveles, al menos 75 por ciento de las cargas verticales están soportadas por muros ligados entre sí mediante losas monolíticas u otros sistemas de piso suficientemente resistentes y rígidos al corte. Dichos muros tendrán distribución sensiblemente simétrica con respecto a dos ejes ortogonales, o en su defecto, el edificio tendrá, en cada nivel, al menos dos muros perimetrales de carga, sensiblemente paralelos entre sí, ligados por los sistemas de piso antes citados en una longitud no menor que la mitad de la dimensión del edificio en la dirección de dichos muros.

II. La relación entre longitud y ancho de la planta del edificio no excede de 2.0 a menos que, para fines de análisis sísmico, se pueda suponer dividida dicha planta en tramos independientes cuya relación longitud a ancho satisfaga esta restricción y cada tramo se revise en forma independiente en su resistencia a efectos sísmicos.

III. La relación entre la altura y la dimensión mínima de la base del edificio no excede de 1.5 y la altura del edificio no es mayor de 13 m

Además, cuando se use dicho método simplificado, la contribución a la resistencia a fuerza cortante de los muros cuya relación de altura de entrepiso, H, a longitud, L, es mayor que 1.33, se reducirá multiplicandola por el coeficiente $(1.33\ L/H)^2$.

4.2 Resistencia a cargas verticales

4.2.1 Fórmula general

La carga vertical resistente se calculará como:

$$P_R = F_R\, F_E\, f^*_m\, A_T$$

donde

P_R es la carga vertical total resistente de diseño

F_R se tomará como 0.6 para muros confinados o reforzados interiormente de acuerdo con 3.3 o 3.4 y como 0.3 para muros no reforzados.

f^*_m es la resistencia de diseño en compresión de la mampostería

F_E es un factor de reducción por excentricidad y esbeltez que se obtendrá de acuerdo con 4.2.2

A_T es el área de la sección transversal del muro

4.2.2 Factor de reducción por excentricidad y esbeltez

I. Cuando se cumplan los requisitos especificados en los incisos a), b) y c) de 4.1.2, podrá tomarse F_E igual a 0.7 para muros interiores que soporten claros que no difieren en más de 50 por ciento y como 0.6 para muros extremos o con claros que difieran en más de 50 por ciento, y para casos en que la relación entre cargas vivas y cargas muertas de diseño excede de uno.

II Cuando no se cumplan las condiciones del caso I, el factor de reducción por excentricidad y esbeltez se determinará como el menor del que se especifica en el caso I y el que se obtiene con la ecuación siguiente

$$F_E = (1 - 2e'/t)\left[1-(\frac{H'}{30t})^2\right] \qquad (4.1)$$

en que

t es el espesor del muro

e' es la excentricidad calculada para la carga vertical, e_c, más una excentricidad accidental que se tomará igual a t/24

H' la altura efectiva del muro que se determinará a partir de la altura no restringida, H, según el criterio siguiente:

H' = 2H, para muros sin restricción al desplazamiento lateral en su extremo superior

H' = 0.8H para muros limitados por dos losas continuas a ambos lados del muro

H' = H para muros extremos en que se apoyan losas

4.2.3 Efecto de las restricciones a las deformaciones laterales

En casos en que el muro en consideración esté ligado a muros transversales a contrafuertes o a columnas o castillos que restrinjan su deformación lateral, el factor F_E calculado con la ec 4.1 se incrementará sumándole la cantidad $(1-F_E)B$, pero el resultado no será en ningún caso mayor que 0.9.

B es un coeficiente que depende de la separación de los elementos rigidizantes, L', y se obtiene de la tabla siguiente

FACTOR CORRECTIVO, B, POR EFECTO DE LA RESTRICCION DE MUROS TRANSVERSALES

L'/H	1.5	1.75	2.0	2.5	3.0	4.0	5.0
B	0.7	0.6	0.5	0.4	0.33	0.25	0.20

4.2.4 Contribución del refuerzo a la resistencia a cargas verticales

La contribución a la resistencia a carga vertical de castillos y dalas o del refuerzo interior se considerará mediante los incrementos en el esfuerzo resistente en compresión, f^*_m, de la mampostería, permitidos según los incisos 2.4.1d) y e) de estas normas, a menos que mediante ensayes a escala natural se haya demostrado que se justifica un incremento mayor en la resistencia debido a dicho refuerzo.

En muros sometidos a momentos flexionantes significativos, perpendicularmente a su plano, podrá determinarse la resistencia en flexocompresión tomando en cuenta el refuerzo vertical del muro, cuando la separación de éste no exceda de seis veces el espesor del muro.

El cálculo se realizará con el criterio de resistencia en flexocompresión que se especifica para concreto reforzado, y con base en las hipótesis siguientes:

a) La distribución de deformaciones unitarias longitudinales en la sección transversal de un elemento es plana.

b) Los esfuerzos de tensión son resistidos por el refuerzo únicamente.

c) Existe adherencia perfecta entre el refuerzo y el concreto o mortero que lo rodea

d) La sección falla cuando se alcanza, en la mampostería, la deformación unitaria máxima a compresión que se tomará igual a 0 003.

e) A menos que ensayes en pilas permitan obtener mejor determinación de la curva esfuerzo-deformación de la mampostería, ésta se supondrá lineal hasta la falla.

Los efectos de esbeltez se tomarán en cuenta afectando la carga resistente del factor $[1-(\frac{H'}{30t})^2]$, según el inciso 4.2.2.

4.3 Resistencia a cargas laterales

4.3.1 Consideraciones generales

La resistencia a cargas laterales de un muro deberá revisarse para el efecto de la fuerza cortante, del momento flexionante en su plano y eventualmente también de momentos flexionantes debidos a empujes normales a su plano.

Cuando sean aplicables los requisitos del método simplificado de diseño sísmico, ver inciso 4.1.3, la revisión podrá limitarse a los efectos de la fuerza cortante.

4.3.2 Fuerza cortante resistida por la mampostería

La fuerza cortante resistente de diseño se determinará como sigue.

a) Para muros diafragma

$$V_R = F_R(0.85\, v^*\, A_T) \qquad (4.2)$$

b) Para otros muros

$$V_R = F_R(0.5 v^*\, A_T + 0.3 P) \le 1.5 F_R v^*\, A_T \qquad (4.3)$$

en donde

P es la carga vertical que actúa sobre el muro, sin multiplicar por el factor de carga

v^* es el esfuerzo cortante medio de diseño que se determinará según el inciso 2.4.2

El factor de reducción de resistencia, F_R, se tomará como:

0.7 para muros diafragma, muros confinados y muros con refuerzo interior, según se definen en el capítulo 3 de estas normas.

0.4 para muros no confinados ni reforzados

No se considerará incremento alguno de la fuerza cortante resistente por efecto de las dalas y castillos de muros confinados de acuerdo con la sección 3.3. Cuando se coloque refuerzo horizontal en las juntas con las características definidas en la sección 3.3 para muros confinados y en la sección 3.4 para muros con refuerzo interior podrá incrementarse en 25 por ciento la fuerza cortante resistente calculada con la ec 4.3, siempre que la cuantía de refuerzo horizontal, p_h, no sea inferior a 0.0005 ni al valor que resulte de la expresión siguiente

$$p_h = 0.0002\, v^* \left(1 + 0.2\, \frac{P}{v^* A_T}\right) \frac{4200}{f_y}$$

4.3.3 Resistencia a flexocompresión en el plano del muro

La resistencia a flexión y a flexocompresión en el plano del muro se calculará, para muros sin refuerzo, según la teoría de resistencia de materiales suponiendo una distribución lineal de esfuerzos en la mampostería. Se considerará que la mampostería no resiste tensiones y que la falla ocurre cuando aparece en la sección crítica un esfuerzo de compresión igual a f^*_m.

La capacidad a flexión o flexocompresión en el plano de un muro con refuerzo interior o exterior se calculará con un método de diseño basado en las hipótesis estipuladas en el inciso 2.4. En todos los casos la capacidad deberá afectarse del factor de resistencia F_R determinado como se indica al final de este inciso 4.3.3

Para muros reforzados con barras colocadas simétricamente en sus extremos, las fórmulas simplificadas siguientes dan valores suficientemente aproximados y conservadores del momento resistente de diseño

Para flexión simple, el momento resistente se calculará como

$$M_O = F_R\, A_S\, f_y\, d'$$

donde

A_s es el área de acero colocada en el extremo del muro

d' la distancia entre los centroides del acero colocado en ambos extremos del muro

Cuando exista carga axial sobre el muro, el momento resistente de la sección se modificará de acuerdo con la ecuación

$$M_R = M_o + 0.30\, P_u\, d \quad ; \quad \text{si } P_u \le \frac{P_R}{3}$$

$$M_R = (1.5\, M_o + 0.15\, P_R\, d)\left(1 - \frac{P_u}{P_R}\right) \quad ; \quad \text{si } P_u > \frac{P_R}{3}$$

donde

P_u es la carga axial de diseño total sobre el muro, que se considerará positiva si es de compresión

d el peralte efectivo del refuerzo de tensión

P_R la resistencia a compresión axial

F_F se tomará igual a 0.8 si $P_u \leq P_R/3$ e igual a 0.6 en caso contrario

5. CONSTRUCCION

5.1 Materiales

5.1.1 Piezas

Condiciones. Las piezas empleadas deberán estar limpias y sin rajaduras.

Humedecimiento de las piezas. Deberán saturarse previamente a su colocación todas las piezas de barro; las piezas a base de cemento deberán estar secas al colocarse.

5.1.2 Morteros

Mezclado del mortero. La consistencia del mortero se ajustará tratando de que alcance la mínima fluidez compatible con una fácil colocación. Los materiales se mezclarán en un recipiente no absorbente, prefiriéndose, siempre que sea posible, un mezclado mecánico. El tiempo de mezclado, una vez que el agua se agrega, no debe ser menor de 3 minutos.

Remezclado. Si el mortero empieza a endurecerse, podrá remezclarse hasta que vuelva a tomar la consistencia deseada agregándole agua si es necesario.

Los morteros a base de cemento normal deberán usarse dentro del lapso de 2.5 horas a partir del mezclado inicial.

5.1.3 Concretos

Los concretos para el colado de elementos de refuerzo, interiores o exteriores al muro, tendrán la cantidad de agua que asegure una consistencia líquida sin segregación de los materiales constituyentes. El tamaño máximo del agregado será de 1 cm.

5.2 *Procedimientos de construcción*

5.2.1 Juntas

El mortero en las juntas cubrirá totalmente las caras horizontales y verticales de la pieza. Su espesor será el mínimo que permita una capa uniforme de mortero y la alineación de las piezas. El espesor de las juntas no excederá de 1.5 cm.

5.2.2 Aparejo

Las fórmulas y procedimientos de cálculo especificados en estas disposiciones son aplicables sólo si las piezas se colocan en forma cuatrapeada; para otros tipos de aparejo, el comportamiento de los muros deberá deducirse de ensayes a escala natural.

5.2.3 Concreto y mortero

En castillos y huecos interiores se colará de manera que se obtenga un llenado completo de los huecos. El colado de elementos interiores verticales se efectuará en tramos no mayores de 1.5 m a menos que el área del hueco sea mayor de 65 cm², caso en el cual se permitirá el colado en tramos hasta de 3 m, siempre que sea posible comprobar, por aberturas en las piezas, que el colado llega hasta el extremo inferior del elemento.

5.2.4 Refuerzo

El refuerzo se colocará de manera que se asegure que se mantenga fijo durante el colado. El recubrimiento, separación y traslapes mínimos serán los que se especifican para concreto reforzado; para esfuerzo colocado en las juntas regirá lo especificado en la sección 3.4. No se admitirá traslape de barras de refuerzo colocadas en juntas horizontales.

5.2.5 Construcción de muros

En la construcción de muros, además de los requisitos de las secciones anteriores, se cumplirán los siguientes:

La dimensión de la sección transversal de un muro que cumpla alguna función estructural o que sea de fachada no será menor de 10 cm.

Todos los muros que se toquen o crucen deberán anclarse o ligarse entre sí, salvo que se tomen precauciones que garanticen su estabilidad y buen funcionamiento.

Los muros de fachada que reciban recubrimiento de materiales pétreos naturales o artificiales deberán llevar elementos suficientes de liga y anclaje para soportar dichos recubrimientos.

Durante la construcción de todo muro se tomarán las precauciones necesarias para garantizar su estabilidad en el proceso de la obra, tomando en cuenta posibles empujes horizontales, incluso viento y sismo.

En los planos de construcción deberán especificarse claramente: peso máximo admisible de las piezas, resistencia de las mismas y tolerancia en sus dimensiones; así como el mortero considerado en el diseño y los detalles del aparejo de las piezas, del refuerzo y su anclaje y traslape, detalles de intersecciones entre muros y anclajes de elementos de fachada.

5.2.6 Tolerancias

a) En ningún punto el eje de un muro que tenga función estructural distará más de 2 cm del de proyecto.

b) El desplome de un muro no será mayor que 0.004 veces su altura ni 1.5 cm.

6. MAMPOSTERIA DE PIEDRAS NATURALES

6.1 Alcance

Esta sección se refiere al diseño y construcción de cimientos, muros de retención y otros elementos estructurales de mampostería del tipo conocido como de tercera. o sea formado por piedras naturales sin labrar unidas por mortero.

6.2 Materiales

6.2.1 Piedras

Las piedras que se empleen en elementos estructurales deberán satisfacer los requisitos siguientes:

Resistencia mínima a compresión en dirección normal a los planos de formación 150 kg/cm²

Resistencia mínima a compresión en dirección paralela a los planos de formación 100 kg/cm²

Absorción máxima 4%

Resistencia al intemperismo: máxima pérdida de peso después de 5 ciclos en solución saturada de sulfato de sodio 10%

Las propiedades anteriores se determinarán de acuerdo con los procedimientos indicados en el capítulo CXVII de las Especificaciones Generales de Construcción de la Secretaría de Obras Públicas (1971).

Las piedras no necesitarán ser labradas, pero se evitará en lo posible el empleo de piedras de formas redondeadas y de cantos rodados. Por lo menos el 70% del volumen del elemento estará constituído por piedras con un peso mínimo de 30 kg cada una.

6.2.2 Morteros

Los morteros que se empleen para mampostería de piedras naturales deberán cumplir con los requisitos siguientes:

a) La relación volumétrica entre la arena y la suma de cementantes se encontrará entre 2.25 y 5.

b) La resistencia mínima en compresión será de 15 kg/cm².

c) La resistencia se determinará según lo especificado en la norma NOM C 61

6.3 Diseño

6.3.1 Esfuerzos resistentes de diseño

Los esfuerzos resistentes de diseño en compresión, f^*_m, y en cortante, v^*, se tomarán como sigue:

Mampostería unida con mortero de resistencia en compresión no menor que 50 kg/cm²

$f^*_m = 20$ kg/cm², $v^* = 0.6$ kg/cm²

Mampostería unida con mortero de resistencia en compresión menor que 50 kg/cm²

$f^*_m = 15$ kg/cm²; $v^* = 0.4$ kg/cm²

Los esfuerzos de diseño anteriores incluyen ya un factor de reducción, F_R, que por lo tanto no deberá ser considerado nuevamente en las fórmulas de predicción de resistencia

6.3.2 Determinación de la resistencia

Se verificará que en cada sección la fuerza normal actuante de diseño no exceda la fuerza resistente dada por la expresión

$$P_R = (1 - 2\,e/t)\,A_t\,f^*_m$$

siendo t el peralte de la sección, A_t, su área y e la excentricidad con que actúa la carga. La expresión anterior es válida cuando la relación entre la altura del elemento de mampostería y el peralte de su sección no excede de 5; cuando dicha relación se encuentre entre 5 y 10, la resistencia se tomará igual al 80% de la calculada con la expresión anterior; cuando la relación exceda de 10 deberán tomarse en cuenta explícitamente los efectos de esbeltez en la forma especificada para mampostería de piedras artificiales.

La fuerza cortante actuante no excederá de la resistente obtenida de multiplicar el área transversal de la sección más desfavorable por el esfuerzo cortante resistente según el inciso anterior.

6.4 Construcción

6.4.1 Piedras

Las piedras que se emplean deberán estar limpias y sin rajaduras. No se emplearán piedras que presentan forma de laja. Las piedras se mojarán antes de usarlas.

6.4.2 Mortero

El mortero se elaborará con la cantidad de agua mínima necesaria para obtener una pasta manejable. Para el mezclado y remezclado se respetarán los requisitos del inciso 5.1.2

6.4.3 Procedimiento constructivo

La mampostería se desplantará sobre una plantilla de mortero o concreto que permita obtener una superficie plana. En las primeras hiladas se colocarán las piedras de mayores dimensiones y las mejores caras de las piedras se aprovecharán para los paramentos. Cuando las piedras sean de origen sedimentario se colocarán de manera que los lechos de estratificación queden normales a la dirección de las compresiones. Las piedras deberán humedecerse antes de colocarlas y se acomodarán de manera de llenar lo mejor posible el hueco formado por las otras piedras. Los vacíos se rellenarán completamente con piedra chica y mortero. Deberán usarse piedras a tizón, que ocuparán por lo menos una quinta parte del área del paramento y estarán distribuídas en forma regular. Se respetarán, además los requisitos del inciso 5.2.5, que sean aplicables.

6.5 Cimientos

En cimientos de piedra braza la pendiente de las caras inclinadas, medida desde la arista de la dala o muro, no será menor que 1.5 (vertical) : 1 (horizontal).

En cimientos de mampostería de forma trapecial con un talud vertical y el otro inclinado, tales como cimientos de lindero, deberá verificarse la estabilidad del cimiento a torsión. De no efectuarse esta verificación, deberán existir cimientos perpendiculares a ellos a separaciones no mayores de las que señala la siguiente tabla:

Presión de contacto con el terreno, p, ton/m2	Claro máximo, en m	
	Caso(1)	Caso (2)
$p\leq2.0$	5.0	10.0
$2.0<p\leq2.5$	4.5	9.0
$2.5<p\leq3.0$	4.0	7.5
$3.0<p\leq4.0$	3.0	6.0
$4.0<p\leq5.0$	2.5	4.5

En todo cimiento deberán colocarse dalas de concreto reforzado, tanto sobre los cimientos sujetos a momentos de volteo como sobre los perpendiculares a ellos. Los castillos deben empotrarse en los cimientos no menos de 40 cm.

En la tabla anterior, el claro máximo permisible se refiere a la distancia entre los ejes de los cimientos perpendiculares menos el promedio de los anchos medios de éstos. Los casos (1 y (2) corresponden respectivamente a mampostería ligada con mortero de cal y con mortero de cemento. No deberán existir planos definidos de falla transversales al cimiento

6.6 Muros de contención

En el diseño de muros de contención se tomará en cuenta la combinación más desfavorable de cargas laterales y verticales debidas a empuje de tierras, al peso propio del muro, a las demás cargas muertas que puedan obrar y a la carga viva que tienda a disminuir el factor de seguridad contra volteo o deslizamiento.

México D.F., a 24 de febrero de 1995, el Jefe del Departamento del Distrito Federal, *Oscar Espinosa Villarreal*.- Rúbrica - El Secretario de Obras y Servicios, *Daniel Ruiz Fernández*.- Rúbrica

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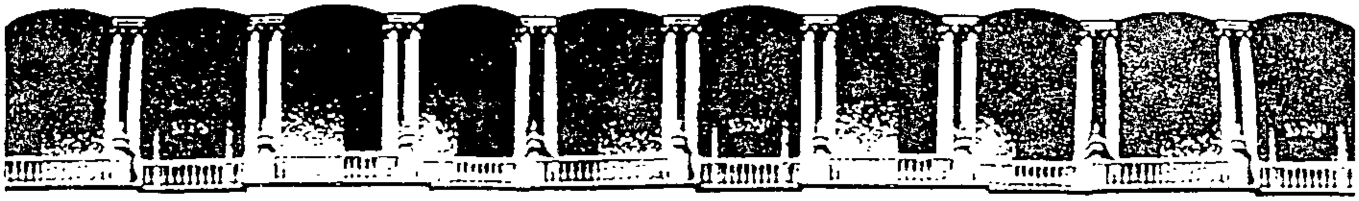

FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

XXVI CURSO INTERNACIONAL DE INGENIERIA SISMICA

MÓDULO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

CRITERIOS DE DISEÑO SISMICO

EXPOSITOR: DR. AMADOR TERAN GILMORE
PALACIO DE MINERIA
AGOSTO DE 2000

CRITERIOS DE DISEÑO SÍSMICO

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La respuesta dínamica de una estructura durante una excitación sísmica depende de las características mecánicas de la primera, y de la manera en que estas interactuan con las características dinámicas de la segunda. Dadas las complejidades e incertidumbres involucradas, no es fácil plantear ni entender el comportamiento y desempeño que una estructura pueda llegar a tener durante una excitación sísmica. Por tanto, el diseño sísmico no puede verse como una ciencia exacta, sino mas como un arte, donde la intuición y buen juicio tienen más que aportar que un cálculo numérico.

Dado lo complejo del tema, en la presentación que se ofrece a continuación se simplifican algunos conceptos con la intención de clarificarlos, y enfatizar algunos detalles que son de importancia. Cuando es necesario profundizar en algunos conceptos, estos se ilustran a través de ejemplos que involucran elementos o estructuras de concreto reforzado. Sin embargo, la presentación es lo suficientemente general para que estos conceptos puedan extenderse con facilidad a otro tipo de materiales estructurales.

FORMULACIÓN DEMANDA-SUMINISTRO

Nuestro enfoque actual de diseño sísmico está basado en una formulación de demanda y suministro:

$$\text{DEMANDA SÍSMICA} \leqslant \text{SUMINISTRO SÍSMICO} \qquad (1)$$

Este enfoque implica primero, estimar las demandas sísmicas, para luego estimar los suministros sísmicos correspondientes. La información que se requiere para establecer la Ecuación 1 puede agruparse en 4 categorías.

- **Criterios de desempeño (comportamiento deseado)**. Es necesario considerar el comportamiento deseado de la estructura durante los diferentes niveles de excitación sísmica considerados relevantes.

- **Niveles sísmicos de diseño.** Es necesario establecer excitaciones sísmicas de diferente intensidad para definir en contra de que se va a diseñar a la estructura. Es importante considerar que la estructura estará sujeta, a lo largo de su vida útil, a excitaciones sísmicas de diferente intensidad.

- **Demandas sísmicas.** Primero, es necesario identificar cuales son las demandas sísmicas relevantes para cada uno de los niveles sísmicos considerados; para esto es importante considerar el comportamiento deseado de la estructura para cada uno de estos niveles. Una vez identificadas, es necesario cuantificar las demandas sísmicas por medio de un análisis estructural.

- **Suministros sísmicos**. Una vez establecidas las demandas, puede entonces establecerse una serie de suministros sísmicos que las satisfagan. Debe enfatizarse que la predicción de las demandas y la evaluación de los suministros sísmicos no es una tarea fácil. Las demandas sísmicas en una estructura dependen fuertemente de sus suministros sismicos (la respuesta de la estructura depende de las características mecánicas que se le suministran), mientras que los suministros sísmicos se proveen a la estructura en función de las demandas estimadas. Esto hace que el diseño sísmico sea iterativo por naturaleza.

CRITERIOS DE DESEMPEÑO/NIVELES SÍSMICOS DE DISEÑO

Actualmente, la filosofía mundial del diseño sísmico se basa en que las estructuras de ocupación estándar deben satisfacer las siguientes condiciones:

- Resistir sin daño (estado límite de servicio) niveles menores de movimiento sísmico;
- Resistir sin daño estructural, aunque posiblemente con algún tipo de daño no estructural (estado límite de operación), niveles moderados de movimiento sísmico;
- Resistir sin colapso, aunque con algún tipo de daño estructural y no estructural ((estado límite de seguridad), niveles mayores de movimiento sismico

Lo anterior implica combinar tres criterios de desempeño (o estados límite), formulados en función del daño estructural y no estructural aceptable, con tres niveles sísmicos de diseño (menor, moderado y mayor). El definir la correspondencia que hay entre los criterios de desempeño y los niveles sísmicos considerados da lugar a los objetivos de diseño.

Como podrá apreciarse, el tercer objetivo de diseño acepta daño en los elementos estructurales, lo que implica que es posible que la estructura vaya mas alla de su rango elástico, y exhiba un comportamiento plástico de importancia. Esto a su vez implica que es posible utilizar suministros de resistencia que, dependiendo de la capacidad de deformación de la estructura, pueden llegar a ser varias veces menor que la resistencia requerida para mantenerla en el rango elástico. La posibilidad de diseñar para estas resistencia reducidas se ha reflejado en mayor racionalidad económica durante el diseño sismorresistente.

CARACTERÍSTICAS MECÁNICAS RELEVANTES

Existen cuatro propiedades de una estructura que tienen mucha relevancia en su comportamiento dinámico durante una excitación sísmica. Tres de estas, la resistencia lateral, la rigidez lateral y la capacidad de deformación son características mecánicas de la estructura que deben diseñarse; mientras que la cuarta, la masa reactiva, normalmente no se diseña, sino que suele conocerse antes del diseño de la estructura. La Figura 1 ofrece una interpretación gráfica de las tres características mecánicas mencionadas arriba. En esta presentación se tomará al cortante basal (V_b), tal como se define en la Figura 1a, como una medida de la resistencia lateral de la estructura.

Como ilustra la Figura 1, cuando se hable de la deformación de la estructura se hará referencia al desplazamiento lateral que en su nivel superior producen las fuerzas laterales inducidas por la excitación sísmica (δ_{azotea}). De alguna manera, y aunque esta no representa una cuantificación ideal, se ha llegado al consenso de que el nivel de deformación puede cuantificarse por medio del concepto de ductilidad de desplazamiento global, μ. La Figura 1b ilustra la definición de ductilidad última μ_u, que es igual al desplazamiento de azotea último que alcanza la estructura cuando se le sujeta a un estado de deformación monotonicamente creciente (δ_u), normalizado por el desplazamiento de azotea de fluencia (δ_y). Como se muestra se recurre a idealizar el comportamiento de la estructura por medio de una curva bilineal para hacer posible la definición de δ_y.

Aunque es necesario formalizar la definición de ductilidad y racionalizar su uso dentro del diseño sismorresistente, el concepto de ductilidad nos permite distinguir entre la capacidad de deformación elástica de una estructura y su capacidad de deformación plástica. No exceder la primera implica que no habrá daño estructural, mientras que un desplazamiento creciente en el rango plástico de comportamiento, caracterizado por la fluencia del material estructural, implica daño creciente. Por tanto, imponer un límite máximo para el desplazamiento lateral que una estructura puede alcanzar durante una excitación sísmica, implica un entendimiento claro de sus capacidades de deformación elástica y plástica, o en otras palabras, de la ductilidad máxima que esta puede desarrollar.

La rigidez de una estructura de concreto reforzado en su rango elástico (K_e) depende del claro y dimensiones transversales de los elementos estructurales, así como del refuerzo longitudinal de los mismos. En el rango inelástico, la rigidez lateral (K_i) depende de varias cosas, dentro de las cuales destacan la regularidad de la estructura, el nivel de cargas gravitacionales y el detallado de los elementos estructurales. La resistencia lateral de una estructura de concreto reforzado depende de las dimensiones de los elementos estructurales y, si esta bien diseñada (el acero longitudinal fluye antes de que se presente cualquier tipo de falla frágil), de la cuantía de acero longitudinal de dichos elementos. Finalmente, la capacidad última de deformación depende del detallado, dimensiones, claro, y resistencia de los elementos estructurales y sus conexiones. Si los elementos no estructurales estan conectados a la estructura, y son de naturaleza tal (por ejemplo muros de relleno de mampostería, que son muy rígidos y resistentes) que afectan de manera importante las características mecánicas de esta, es necesario tomarlos en cuenta cuando se establecen las características mecánicas de la estructura.

Puede notarse que el valor de las características mecánicas mencionadas en el párrafo anterior dependen del valor de varios parámetros comunes, de manera que existe una interacción importante entre estas. Un cambio en el valor de una de estas características mecánicas afecta necesariamente el valor de las otras. A pesar de esto, su dependencia no es de tal naturaleza que pueda establecerse un relación directa entre ellas, de manera que puedan obviarse, durante el diseño sísmico, algunas de ellas en favor de otras. Esto implica que durante el diseño sísmico deben tomarse en cuenta explicitamente cada una de estas tres características mecánicas. Dentro de este contexto, es importante reconocer que no hay ni material ni sistema estructural ideal para tomar las acciones que las excitaciones sísmicas inducen en las estructuras. Cada material y sistema estructural ofrece una combinación de estas tres características mecánicas que exhibe fortalezas y debilidades para resistir las excitaciones sísmicas.

Con base a lo discutido arriba es conveniente plantear la ecuación demanda-suministro conforme:

DEMANDA SÍSMICA $\leq$ SUMINISTRO SÍSMICO (2)

DEMANDA SÍSMICA de	SUMINISTRO SÍSMICO de
Resistencia	Resistencia
Rigidez	Rigidez
Capacidad de deformación	Capacidad de deformación

Como se ilustra en la Figura 1b, K_e establece en el rango elástico una relación entre la fuerza lateral total actuando en la estructura y la deformación lateral que la primera induce en la segunda. La rigidez lateral de la estructura no solo debe verse como una medida para controlar la deformación lateral, sino como una propiedad que determina en gran medida la respuesta de la estructura ante una excitación sísmica. Para entender esto, considere que el periodo de la estructura depende directamente de su rigidez, y que la respuesta dinámica de la estructura depende fuertemente de su periodo. Esto puede entenderse a partir del concepto de espectro, que se ilustra en la Figura 2. La Figura 3 muestra espectros obtenidos para El Centro NS 1940 y SCT EO 1985, excitaciones sísmicas características de suelo firme y blando, respectivamente. La abcisa de las gráficas presentadas en las Figuras 2 y 3 corresponde al periodo, mientras que las ordenadas corresponden a la resistencia mínima que requiere un sistema de un grado de libertad con un periodo dado, para permanecer en su rango elástico de comportamiento durante la excitación sísmica. Puede notarse que las demanda de resistencia depende de manera importante en el valor de periodo y, por tanto, en el valor de la rigidez lateral de la estructura.

Para una rigidez lateral dada, esto es, para un periodo T dado, la resistencia lateral de la estructura exhibe una fuerte influencia en su demanda máxima de desplazamiento plástico (recuerde que en esta presentación se caracterizará dicha demanda a través de la demanda máxima de ductilidad). La Figura 4 ilustra esquematicamente la interacción que existe entre la resistencia lateral y la demanda de μ. Como se ilustra, un incremento en la resistencia lateral disminuye de manera importante la demanda de μ, particularmente cuando este incremento se da a partir de valores muy bajos de resistencia. Para un periodo dado, un incremento de resistencia disminuye las demandas plásticas de deformación, tanto máximas como acumuladas, y por tanto se ve reflejado en una disminución en el nivel de daño estructural. Note que, como se ilustra en la Figura 4, para un T dado existen diferentes resistencias asociados a diferentes valores de μ. Con los diferentes valores de resistencia, pueden plantearse espectros inelásticos de resistencia, como los incluidos en la Figura 5, que resumen las resistencias que deben tener sistemas con diferente T para que su demanda máxima de μ durante la excitación sísmica sea igual al valor de μ asociado al espectro.

A partir de los puntos A y B ilustrados en la Figura 4, pueden plantearse dos criterios de diseño sísmico (el punto A esta asociado a demandas de $\mu < 2$, mientras que el punto B esta asociado a demandas altas de μ, digamos del orden de 6):

- Punto A. Las demandas máximas y acumuladas de comportamiento plástico inducidas en estructuras con alta resistencia suelen ser bajas. Por lo general estas estructuras presentan elementos estructurales robustos con cantidades importantes de refuerzo longitudinal. El detallado del refuerzo transversal y otros aspectos de detallado (por ejemplo, traslapes y anclaje) no se cuidan mucho, de manera que los elementos estructurales en particular, y la estructura en general, exhiben poca capacidad de deformación plástica. Este tipo de sistemas basa su supervivencia durante la excitación sísmica practicamente en su capacidad resistente. Si esta se excede, normalmente ocurren fallas altamente indeseables y catastróficas; lo que implica que el diseño de su resistencia debe ser muy conservador.

- Punto B. Las estructuras con baja resistencia lateral suelen sufrir demandas máximas y acumuladas de comportamiento plástico muy elevadas cuando se les sujeta a excitaciones sísmicas severas. Estas estructuras presentan por lo general elementos estructurales esbeltos con cantidades relativamente bajas de refuerzo longitudinal. Es muy importante refinar el detallado de los elementos estructurales (estribos cerrados y cercanos, anclaje, etc.) para que estos sean capaces de incursionar de manera importante en su rango plástico de comportamiento. Esta alta capacidad de deformación es importante no solo para evitar la falla de los elementos estructurales, sino el deterioro progresivo de sus caracteristicas mecánicas debido a un fenómeno de fatiga de bajo ciclaje. Es importante recalcar que cuando un sistema entra, durante una excitación sísmica severa, de manera importante a su rango de comportamiento plástico, es difícil predecir con precisión su respuesta dinámica

Un gran porcentaje de estructuras sismorresistentes se diseña actualmente con un criterio intermedio entre el A y el B. Este criterio intermedio enfatiza la importancia de lograr un equilibrio entre las capacidades resistente y de deformación última que se le suministran a la estructura. De esta manera no solo se logra una solución economicamente factible, sino confiable desde un punto de vista estructural. Este criterio intermedio resulta en elementos estructurales algo robustos con un buen detallado sísmico; los cuales sufrirán demandas moderadas de comportamiento plástico durante excitaciones sísmicas severas. Bajo estas circunstancias, es posible predecir, con un grado de precisión aceptable, la respuesta dinámica de la estructura.

MANEJO DE LA RESISTENCIA LATERAL

Se discutió antes que un incremento en la resistencia lateral de una estructura sismorresistente resulta en una disminución de sus demandas de deformación plástica, y por tanto, en su nivel de daño estructural. Dicho en otras palabras, un incremento de resistencia suele reflejarse en un mejor desempeño estructural. Sin embargo, un incremento de resistencia no siempre se ve reflejado en un mejor desempeño sísmico del contenido de la estructura o de sus elementos no estructurales. En algunos casos, un incremento de resistencia viene acompañado con incrementos importantes de la aceleración o distorsión de entrepiso, lo que resultaría en un mayor nivel de daño en el contenido de la estructura (equipo, inmobiliario, instalaciones, etc.) y de los elementos no estructurales, respectivamente.

Anexo a estas notas se incluye el artículo titulado *Efecto de la resistencia en las diferentes demandas sísmicas* donde esto se discute en detalle.

ANÁLISIS ESTRUCTURAL

Dentro del enfoque actual de diseño sísmico, se considera que se conoce la capacidad última de deformación plástica que la estructura es capaz de alcanzar cuando se le sujeta a un estado de deformación monotonicamente creciente. Como se discutió antes (ver Figura 1b), esta capacidad se caracterizará por medio de μ_u. El diseño sísmico consiste entonces en determinar la resistencia lateral y rigidez lateral que deben proporcionarse a la estructura para que, durante la excitación sísmica de diseño asociada al estado límite de seguridad, su demanda máxima de ductilidad no exceda μ_u. De tal manera que el análisis estructural se plantea a partir de la siguiente formulación parcial de la ecuación demanda-suministro:

DEMANDA SÍSMICA $\leq$ SUMINISTRO SISMICO (2)

de de

Resistencia Resistencia
Rigidez Rigidez

Dado que normalmente se utilizan métodos de análisis elástico para resolver el análisis estructural, se plantea una relación lineal entre la resistencia lateral y la rigidez lateral de la estructura, que no es válida en el rango plástico de comportamiento mostrado en la Figura 1b. Otro aspecto por considerarse es que la ecuación 2 se plantea solo para niveles mayores de excitación sísmica, y se supone que el diseño de la estructura resistente bajo esta condición resulta en estructuras capaces de satisfacer los criterios de desempeño asociados a niveles menor y moderado de excitación sísmica.

La capacidad de deformación de la estructura no se maneja explicitamente, sino a través del detallado de la estructura. Los códigos exigen ciertos requerimientos de detallado (separación y remate de los estribos, anclaje, etc.) que se asocian con una μ_u dada. Esta μ_u a su vez se asocia con un factor de reducción de resistencia, R_μ, a partir del cual pueden establecerse espectros inelásticos de resistencia, como se ilustra en la Figura 6. A partir de estos espectros, es posible definir fuerzas laterales concentradas al nivel de las losas de entrepiso, como muestra la Figura 7a, para modelar el efecto de la excitación sísmica de diseño sobre la estructura. Estas fuerzas inducen elementos mecánicos en los elementos estructurales, y producen distorsiones de entrepiso, que constituyen una medida de las demandas de resistencia y rigidez en la estructura, respectivamente. Los elementos mecánicos (axial, cortante, momento flexionante, etc.) inducidos en los elementos estructurales constituyen las demandas de resistencia que deberán satisfacerse mediante un refuerzo longitudinal y transversal adecuado. La distorsión de entrepiso (definida en la Figura 7b) da una medida de las demandas de rigidez, ya que los reglamentos especifican limites de distorsión máxima que no deben excederse durante el análisis estructural. El excederlos implica aumentar el tamaño de los elementos estructurales (esto es, su rigidez) hasta que se cumpla con dichos límites.

Es importante enfatizar que el análisis estructural no debe utilizarse directamente para concebir el sistema estructural, sino para reforzar o replantear la concepción inicial de la estructura (que debe plantearse antes de pasar a la etapa númerica del diseño). Como se discutirá mas adelante, particularmente en el tema de DISEÑO POR CAPACIDAD, antes de llevar a cabo el análsis estructural el proyectista tiene que tener una idea muy clara de como la estructura que concibe debe resistir las acciones que en ella induce la excitación sísmica de diseño. Esto es particularmente válido en situaciones, como lo es la del diseño sísmico, donde la estimación de las cargas de diseño y de la respuesta de la estructura ante estas es altamente incierta. Bajo este contexto, los resultados del análisis estructural deben ser subordinados a un planteamiento conceptual sólido, que considere otro tipo de medidas de naturaleza no númerica que fomenten el desempeño adecuado de la estructura.

A partir del estudio de la Figura 8 pueden vislumbrarse algunas de las limitaciones de un análisis estructural elástico. En esta figura se contrasta las respuestas elástica e inelástica de un edificio de 10 pisos. Como puede apreciarse, tan pronto como el edificio entra en su rango plástico de comportamiento, se presenta una acumulación muy importante de deformación en los pisos inferiores. Esta acumulación, que es importante para una estructura regular como la mostrada en la Figura 8, puede resultar imposible de predecir en estructuras con irregularidades de importancia, tanto en planta como elevación, de resistencia, rigidez y masa reactiva.

CONTEXTO DEL DISEÑO SÍSMICO

Es práctica común que los códigos prescriban métodos simplificados para estimar las demandas sísmicas en la estructura sismorresistente. Por ejemplo, una suposición típica con el fin de estimar las distorsiones de entrepiso (a partir de las cuales se determina la rigidez lateral de la estructura) es que el desplazamiento lateral de la estructura durante la excitación sísmica de diseño, es independiente de su resistencia lateral. Esto es, el desplazamiento que tendría la estructura si incurre en su rango plástico de comportamiento es exactamente igual al que tendria si permaneciera elástica. Esto, como se discute en detalle en el artículo titulado *Efecto de la resistencia en las diferentes demandas sísmicas*, no sucede en muchos casos de interés práctico (por ejemplo, el desplazamiento de estructuras con periodo fundamental de vibración pequeño es muy sensible a la resistencia lateral de las mismas; lo mismo que para estructuras desplantadas sobre suelo blando y cuyo T se aproxima al periodo dominante de la excitación). A lo anterior, es necesario añadir el hecho de que, como se ilustra en la Figura 9, muchas veces el espectro de diseño, y los espectros de respuesta obtenidos a partir de excitaciones generadas en el sitio de la construcción presentan diferencias de importancia. Lo anterior implica dos cosas:

- El proyectista tiene a su disposición un espectro de diseño elástico que no necesariamente refleja todas las características importantes de las excitaciones sísmicas a las que estará sujeta la estructura.

- La manera sobresimplificada con que el proyectista estima la demanda de desplazamientos y de comportamiento no lineal (demanda de ductilidad) en la estructura no siempre da resultados razonables.

Por tanto, las demandas de desplazamiento y ductilidad que se dan en la estructura durante una excitación sísmica pueden variar significativamente con respecto a aquellas presumidas por la normatividad vigente.

Además de lo anterior, es necesario reconocer que la incertidumbre inherente en la respuesta sísmica de las estructuras hace que el cálculo de sus demandas de desplazamiento y ductilidad (aún si se contara con un espectro de diseño que incluyera todas las características de los movimientos generados en el sitio de la construcción) da lugar a estimaciones aproximadas de dichas demandas. Esto es, es necesario reconocer que la respuesta sísmica de una estructura, especialmente si entra de manera importante en su rango de comportamiento plástico, es altamente incierta. La Tabla 1 resume las fuentes más importantes de incertidumbre, y una aproximación a su contribución a la incertidumbre total asociada a la predicción de la respuesta sísmica de las estructuras.

Tabla 1 Fuentes de incertidumbre en la respuesta sísmica

Fuente de incertidumbre	Porcentaje
Características e intensidad de la excitación sísmica	50
Respuesta sísmica no lineal de la estructura	30
Modelado	10
Diseño vs Construcción	10
Total	100

Ante semejante panorama, que incluye metodologías sobresimplificadas y alta incertidumbre, es necesario tomar precauciones que permitan garantizar el comportamiento adecuado de las estructuras ante excitaciones sísmicas de alta intensidad. Un parte importante en cuanto a esto es garantizar que el comportamiento de la estructura sea:

- Estable. Sea cual sea el mecanismo sismorresistente seleccionado, es necesario que este funcione a lo largo de toda la excitación sísmica sin exhibir una degradación excesiva de sus propiedades sismorresistentes.

- Consistente. Es necesario fomentar que se active el mismo mecanismo sismorresistente (aquel que ha sido seleccionado) independientemente del tipo de excitación sísmica al que se sujeta a la estructura. Por ejemplo, considere el caso en que el proyectista decida que la estructura que diseña debe resistir la excitación(es) sísmica(s) de diseño desarrollando un mecanismo de viga débil-columna fuerte. En este caso, el diseñador deberá tomar las precauciones necesarias para que, independientemente del tipo e intensidad del sismo, el comportamiento no lineal en el edificio tienda a concentrarse en las vigas.

- Controlado. La respuesta dinámica de la estructura debe permanecer dentro de ciertos límites de respuesta que son consistentes con el desempeño estructural y no estructural deseado. En muchos casos es deseable controlar las demandas máximas y acumuladas (en su caso) de aceleración, desplazamiento y comportamiento plástico, para así mantener la integridad del contenido de la estructura, y fomentar el buen desempeño sísmico de sus elementos no estructurales y estructurales.

En cuanto a lo que esto implica para el diseño de la estructura, puede decirse que: " *Es importante que el estructurista limite que es lo que puede hacer la estructura durante la(s) excitación(es) sísmica(s) de diseño, de manera que la excesiva libertad en su respuesta no la conduzca a límites inaceptables de comportamiento*".

A estas alturas surge de manera natural la siguiente pregunta: ¿Que puede hacer el diseñador para fomentar que el comportamiento de la estructura que diseña sea estable, consistente y controlado? Entre las acciones que debe tomar, destacan las siguientes:

- Configuración estructural. Tanto como sea posible, la configuración estructural del edificio debe ser: sencilla, simétrica, regular y redundante. Cabe aclarar que si los elementos no estructurales contribuyen a resistir las cargas laterales, entonces también deberán tomarse en cuenta para definir la configuración estructural de la estructura.

  - Sencilla. Los mecanismos sismorresistentes deben bajar las acciones sísmicas desde donde se generan hasta el suelo, de una manera clara y sencilla.

  - Simetria. Dentro de lo posible, los elementos resistentes deben ubicarse en planta de manera simétrica con respecto al centro de masa de la estructura. De esta manera, se minimiza la respuesta torsional del edificio, que en algunos casos puede llevar a que las demandas de ductilidad se concentren de manera excesiva, y por tanto peligrosa, en unos cuantos elementos resistentes. Por razones similares, es importante evitar distribuciones asimétricas de masa en planta.

  - Regularidad. Es importante mantener una distribución razonable de resistencia, rigidez y masa en altura (esto es, evitar discontinuidades importantes). Variaciones importantes de estas propiedades en altura pueden resultar en concentraciones excesivas de comportamiento no lineal en los elementos de un piso.

- Redundancia. Es muy recomendable repartir la labor de resistir la excitación sísmica entre varios elementos sismorresistentes.

■ Suministros sísmicos. Las características mecánicas de la estructura deben diseñarse explicitamente, considerando cuidadosamente como su resistencia lateral, rigidez lateral y capacidad de deformación impactan su desempeño sísmico.

■ Detallado. Un buen detallado es esencial para fomentar un comportamiento dúctil en la estructura, así como para evitar que se generen mecanismos frágiles durante la respuesta sísmica de la misma.

DISEÑO POR CAPACIDAD

Para motivar la discusión acerca de la necesidad del diseño por capacidad, se discutirá el ejemplo resumido en la Figuras 10 y 11. El ejemplo consiste en el diseño bajo cargas gravitacionales y laterales de un paso a desnivel que puede idealizarse como un sistema de un grado de libertad. La Figura 10 muestra que la masa del sistema estructural se asume concentrada al nivel del tablero del piso del paso a desnivel. Además, se ilustran las acciones de diseño por concepto de carga gravitacional y sísmica, así como la superposición de estas dos condiciones para obtener las acciones de diseño.

La Figura 11 muestra una serie de suministros sísmicos que satisfacen las demandas sísmicas obtenidas en la Figura 10 Dichos suministros no contradicen los requerimientos ni filosofía de una gran cantidad de códigos actuales. Sin embargo, como se muestra en la Figura 11, el diseño que resulta a partir de ellos esta lejos de tener un comportamiento adecuado cuando se requiere que el paso a desnivel entre a su rango no lineal de comportamiento. En particular, el mecanismo sismorresistente deseable para el paso a desnivel es el mecanismo dúctil definido a partir de la fluencia a flexión del acero en la base de la columna. Para que este mecanismo se forme, es necesario que el cortante basal en la columna alcance el valor de 7.5, de manera que por su brazo de palanca de 20, de un momento en la base de 150. Sin embargo, como se muestra en la Figura 11, la resistencia a corte de la columna es de 6, por lo que la columna exhibiría un falla frágil a corte antes de que pueda formar un mecanismo dúctil. La Figura 11 muestra, suponiendo que la carga gravitacional en el paso a desnivel es igual a la mostrada en la Figura 10, que aun proporcionando una resistencia adecuada a corte a la columna, el mecanismo dúctil deseado no se formaría, ya que antes fallaría la cimentación de la estructura.

El ejemplo anterior, aunque muy simple, demuestra que los suministros sísmicos deben satisfacer ciertas condiciones que no siempre son requeridas por los códigos de diseño sismorresistente. Esto ha llevado a plantear filosofías de diseño, como la de diseño por capacidad, que garanticen el comportamiento adecuado de las estructuras.

El objetivo del la filosofía de diseño por capacidad es producir sistemas estructurales que sean capaces de resistir las excitaciones sísmicas por medio de mecanismos estables, consistentes y controlados. Este concepto normalmente se aplica al diseño de estructuras dúctiles, y se enfoca al planteamiento de un mecanismo dúctil para disipar la energía que el sismo introduce a las mismas.

El correcto uso de cualquier filosofía de diseño sismorresistente empieza por identificar el contexto bajo el cual se da el diseño de la estructura. En particular, existen una serie de limitantes que el diseñador debe tomar en cuenta como punto de partida:

- Criterios de diseño. En la mayoría de los casos, los criterios de diseño de las estructuras estan definidos acorde al las funciones (tipo de ocupación) que debe desempeñar la estructura. Estos criterios suelen expresarse en función del daño estructural y no estructural que es aceptable en la estructura durante los sismos a las que estará sujeta.

- Configuración arquitectónica. El proyecto estructural esta limitado por los requerimientos arquitectónicos.

- Normatividad vigente. Los códigos de diseño sismorresistente imponen requerimientos mínimos de diseño que no pueden ignorarse.

- Excitaciones sísmicas de diseño. Las propiedades del terreno en el sitio de la construcción, y las posibles fuentes sismogénicas que afectan al mismo, determinan las características de las excitaciones sísmicas de diseño.

- Incertidumbre. Como se mencionó antes, existen varias fuentes de incertidumbre inherentes al diseño sismorresistente.

Entre las herramientas con las que el proyectista cuenta, para establecer un buen diseño a partir de las limitantes planteadas, se encuentran:

- Conocimiento. Es deseable que el ingeniero posea un nivel de conocimiento que le permita ir mas allá de la simple aplicación de los requerimientos mínimos planteados por la normatividad vigente.

- Intuición. Ademas, es deseable que el ingeniero sea capaz de intuir la pertinencia de las soluciones que plantea, los cambios que deban hacerse a su planteamiento original, y la confiabilidad de los cálculos que realiza durante la fase numérica del diseño.

- Herramientas de análisis estructural. Actualmente existen poderosas herramientas de análisis estructural que permiten al proyectista estimar, y hasta visualizar, el efecto que la excitación sísmica tiene sobre la estructura que diseña.

- Filosofía/Metodología de diseño. Se debe recurrir a metodologías de diseño basadas en conceptos estructurales sólidos, como es el caso del diseño por capacidad.

Los pasos que deben seguirse para hacer un diseño por capacidad pueden resumirse conforme a lo siguiente:

- Primero, es necesario identificar los posibles modos de comportamiento y falla de la estructura, y establecer entre ellos una jerarquía de ocurrencia. En el ejemplo del paso a desnivel existen tres modos principales: uno estable de disipación de energía, y dos indeseables. El proyectista debe decidir en cual(es) modo(s) es deseable que la estructura responda, y cuales deben evitarse a toda costa. En el ejemplo del paso a desnivel, los modos se clasificarían, de mejor a peor conforme a lo que sigue: fluencia en la base de la columna, falla a corte de la columna, falla de cimentación.

- Segundo, es necesario seleccionar un mecanismo sismorresistente estable y consistente. Esto implica definir el material y sistema estructural, que en el caso del ejemplo planteado consiste en un sistema dúctil de concreto reforzado. Parte esencial de este planteamiento es que se identifiquen las zonas donde se concentrará el comportamiento no lineal en la estructura, que para el paso a desnivel es la base de las columnas.

- Tercero, es necesario fomentar que la estructura responda controladamente a través del mecanismo seleccionado. Esto generalmente se logra por medio de: la selección de configuraciones estructurales adecuadas (que eviten la concentranción excesiva de deformación no lineal en unos cuantos elementos); el diseño contra modos de falla indeseables (proporcionarles suficiente resistencia para que no se produzcan antes de que se active el modo de disipación de energía deseado); y el detallado de las zonas que disipan la energía (para garantizar un suministro adecuado de capacidad de deformación en el rango plástico). En el caso del ejemplo del paso a desnivel, esto implica proporcionar suficiente capacidad a corte a la columna, y suficiente capacidad a la cimentación para que la estructura sea capaz de desarrollar su mecanismo dúctil. Además, esto implicará un detallado adecuado en la base de la columna que le permita acomodar las demandas de deformación no lineal esperadas durante las excitaciones sismicas de diseño.

Uno de los casos mas ilustrativos en cuanto al uso de la filosofía de diseño por capacidad dentro de la normatividad actual es el de diseño de marcos dúctiles de concreto reforzado. Al respecto, cabe mencionar que los requerimientos especificados en el Reglamento de Construcciones del D.F. (RCDF) fomentan un mecanismo estable de disipación de energía por medio de la fluencia a flexión del acero longitudinal. Parte importante de este enfoque consiste en:

- Limitar la relación de esbeltez de los miembros estructurales (especialmente las vigas) para fomentar un comportamiento a flexión en ellos y disminuir lo mas posible los efectos de corte (en la Figura 12 puede verse que L/H se limita a valores mayores de 4, donde L es el claro de la viga y H su peralte total).

- Limitar las cuantías mínima y máxima de acero longitudinal de los elementos estructurales a cantidades que permitan un comportamiento dúctil (ver los requerimientos para A_s y A_s' de las vigas en la Figura 13);

- Proveer acero longitudinal a compresión en cantidad importante para fomentar el comportamiento dúctil de la pieza (por ejemplo, en la Figura 13 se especifica que en vigas la mitad del acero positivo no debe ser menor que el de acero negativo en L_1);
- Proveer confinamiento adecuado al concreto, por medio de estribos, para incrementar su capacidad de deformación y permitir el comportamiento dúctil de los elementos estructurales (ver en la Figura 14 los estrictos requerimientos de separación y detallado de los estribos en la zona L_1 de las vigas).

Note que la zona definida por la longitud L_1 es la zona donde se presume se concentrarán las demandas de comportamiento no lineal, esto es, las zonas críticas de disipación de energía.

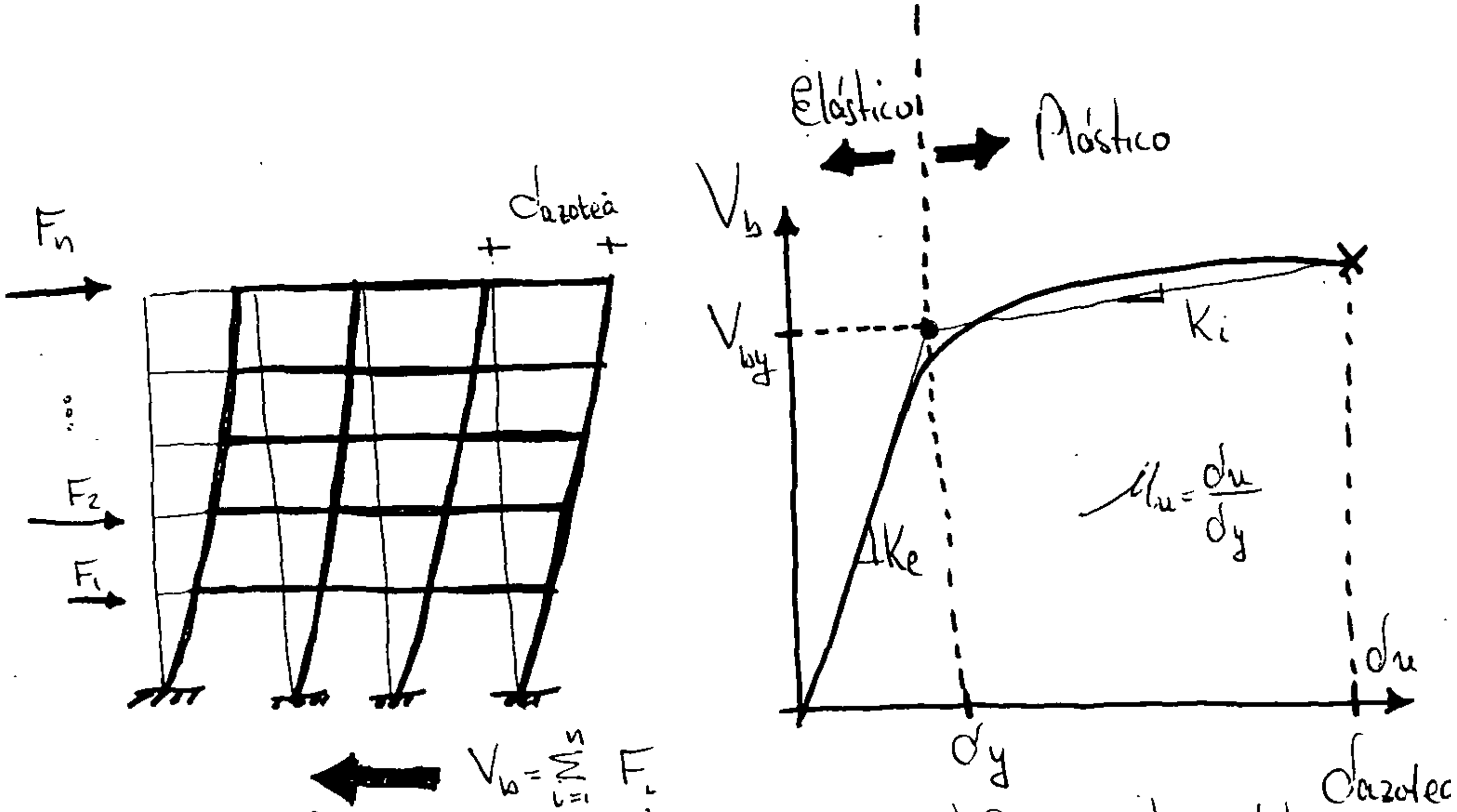
Para fomentar que el mecanismo de disipación de energía sea consistente, el RCDF tiene requerimientos para evitar que ocurran los varios mecanismos frágiles mediante los cuales pueden fallar los diferentes elementos de concreto. A continuación se incluye una lista de estos modos de falla indeseables y algunas de las medidas que toma el RCDF para evitarlas.

- Pandeo lateral de los elementos estructurales. Para evitar el pandeo lateral de los elementos estructurales se limita la relación que su ancho b puede tener con otras dimensiones (ver en la Figura 12 las limitaciones impuestas al valor de b y a sus relaciones con otros elementos).
- Pandeo prematuro del refuerzo longitudinal. Una de las funciones del acero transversal es proporcionar soporte lateral al refuerzo longitudinal para evitar su pandeo (ver en la Figura 14 los estrictos requerimientos de separación y detallado de los estribos en la zona L_1 de las vigas).
- Aplastamiento del concreto en zonas de compresión excesiva. Otra de las funciones del acero transversal es mejorar las propiedades del concreto que se sujeta a compresión para evitar su aplastamiento prematuro (ver en la Figura 14 los estrictos requerimientos de separación y detallado de los estribos en la zona L_1 de las vigas)
- Fractura del acero. La cuantía mínima de acero a tensión especificada en la Figura 13 tiene como objeto el evitar la fractura prematura de este acero.
- Fallas a corte de los elementos estructurales. Las Figuras 15 y 16 ilustran que los requerimientos del RCDF exigen una resistencia a corte en los elementos estructurales que les permita alcanzar la fluencia a flexión en sus dos extremos.
- Formación simultanea de articulaciones plásticas en columnas. La Figura 16 ilustra que los requerimientos del RCDF fomentan un mecanismo columna fuerte-viga débil por medio de proporcionar una resistencia a flexión en las columnas bastante mayor a la correspondiente a las vigas.

- Conexiones o nudos. Se promueve que el acero de las vigas fluya antes que falle la conexión viga-columna, por medio del suministro, en cantidad suficiente, de estribos en la zona de la conexión (ver Figura 17).

- Anclaje/Adherencia. Un aspecto muy importante para permitir la fluencia del acero longitudinal de los elementos estructurales es evitar las posibles fallas de adherencia o anclaje por medio de los estrictos requerimientos mostrados en la Figuras 18 y 19.

Cabe aclarar que aquí no se ha realizado un examen exhaustivo de los requerimientos del Capítulo 5 de las Normas de Concreto del RCDF. Solo se han ilustrado como algunos de estos forman parte de una filosofía de un diseño de capacidad. También es importante mencionar que esta filosofía también va implícita en los requerimientos para el diseño de muros dúctiles de concreto reforzado.



a) Parámetros de resistencia y deformación

b) Curva resistencia lateral contra deformación lateral

Figura 1 Características mecánicas

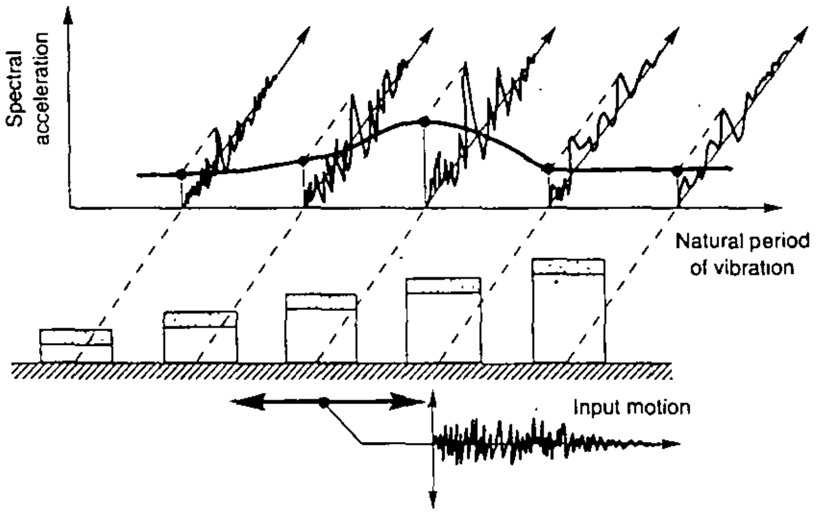
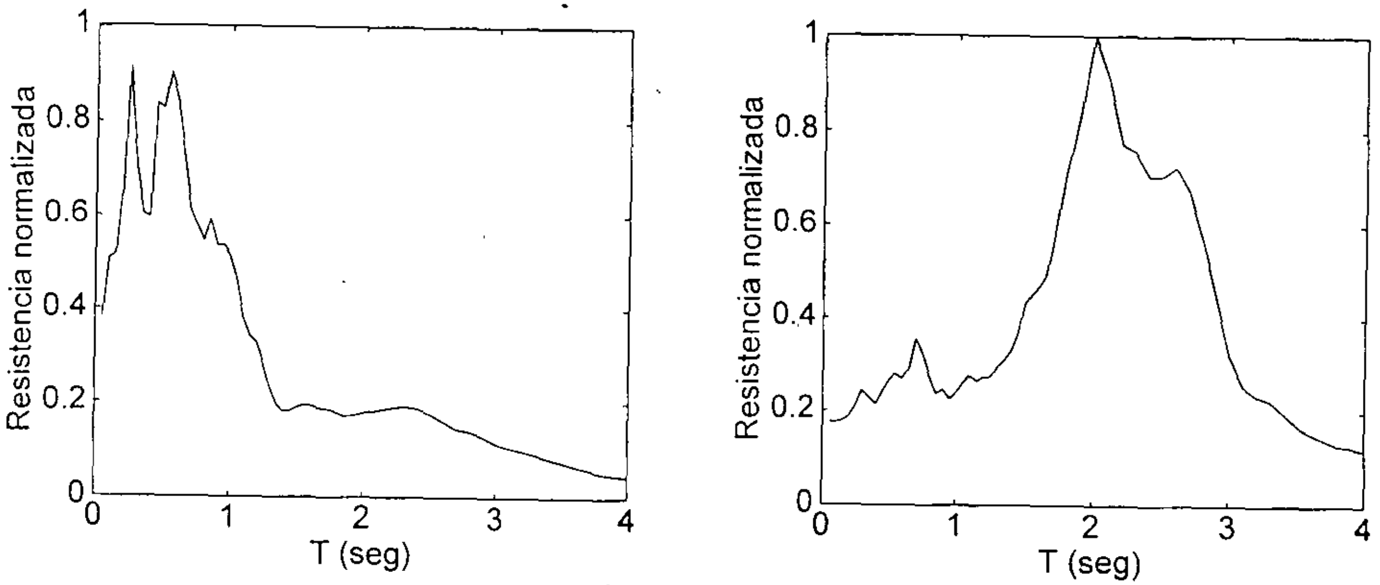


Figura 2 Concepto de espectro



a) El Centro NS 1940

b) SCT EO 1985

Figura 3 Espectros elásticos de resistencia

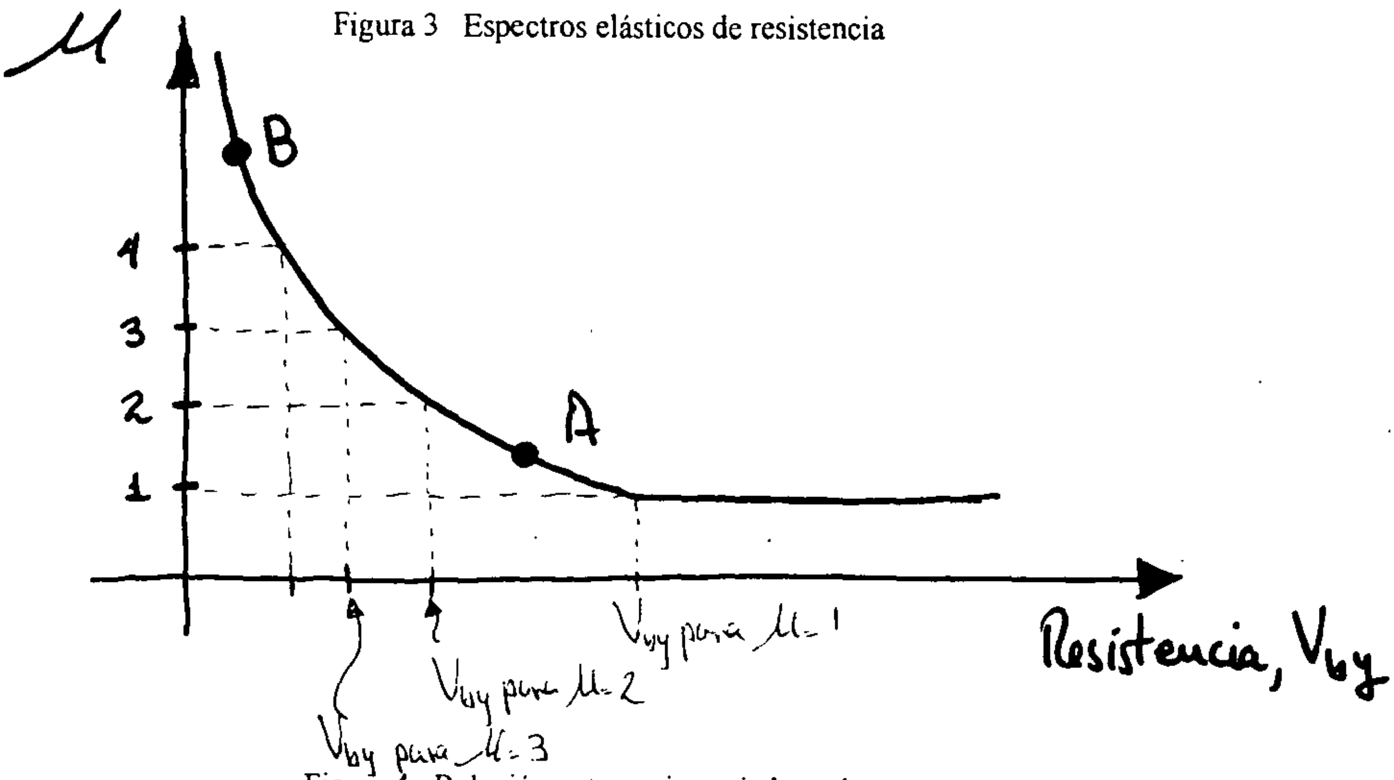
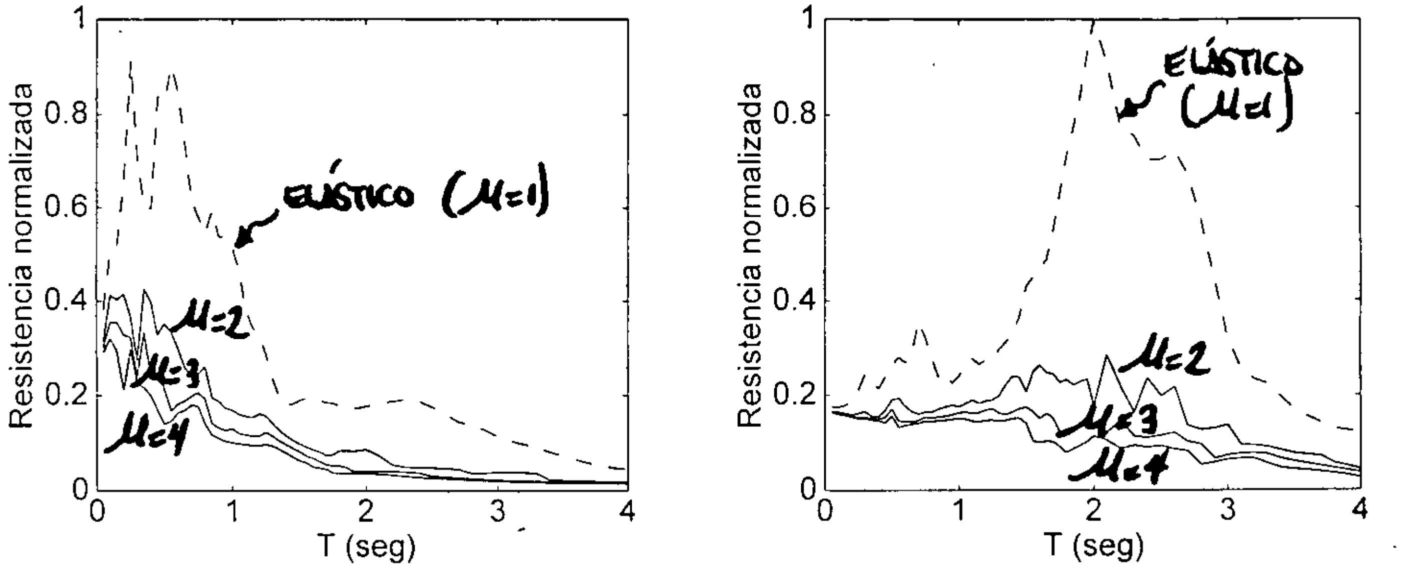


Figura 4 Relación entre resistencia lateral y μ



a) El Centro NS 1940

b) SCT EO 1985

Figura 5 Espectros inelásticos de resistencia

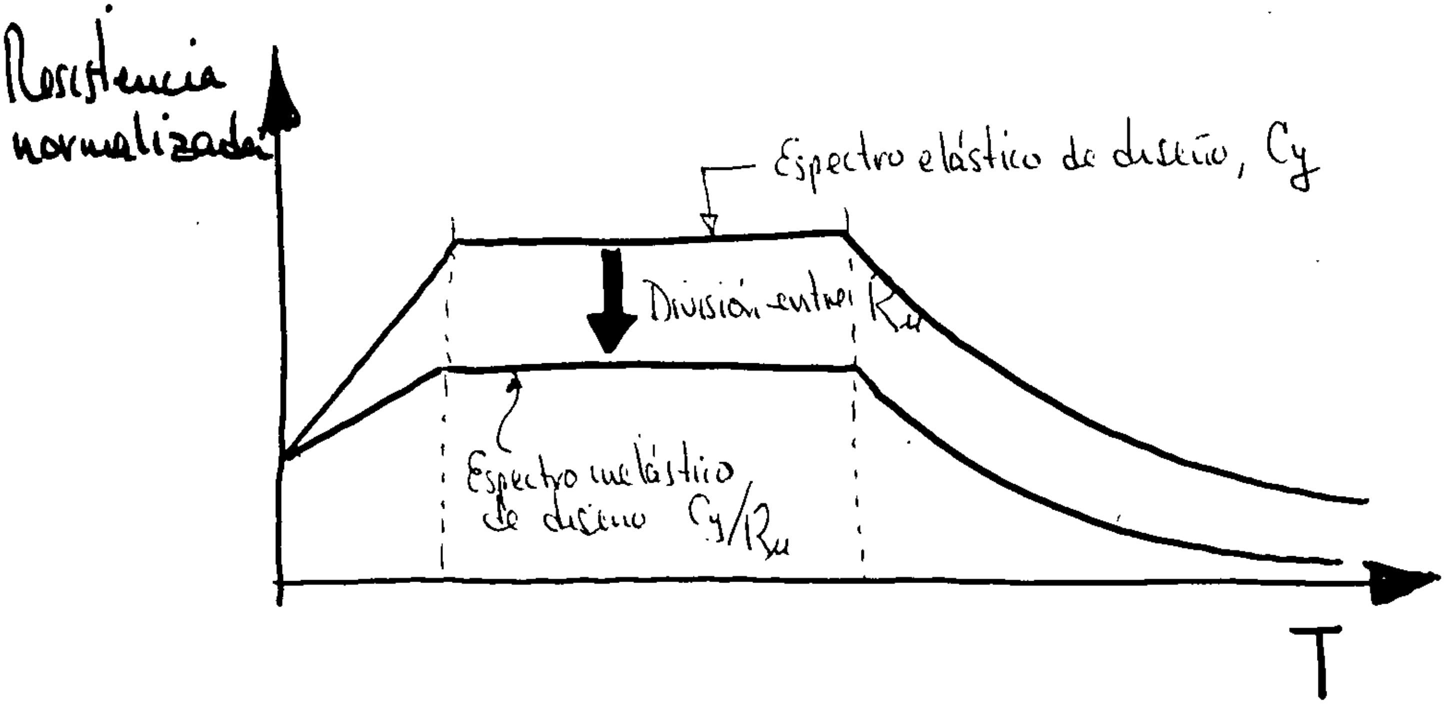
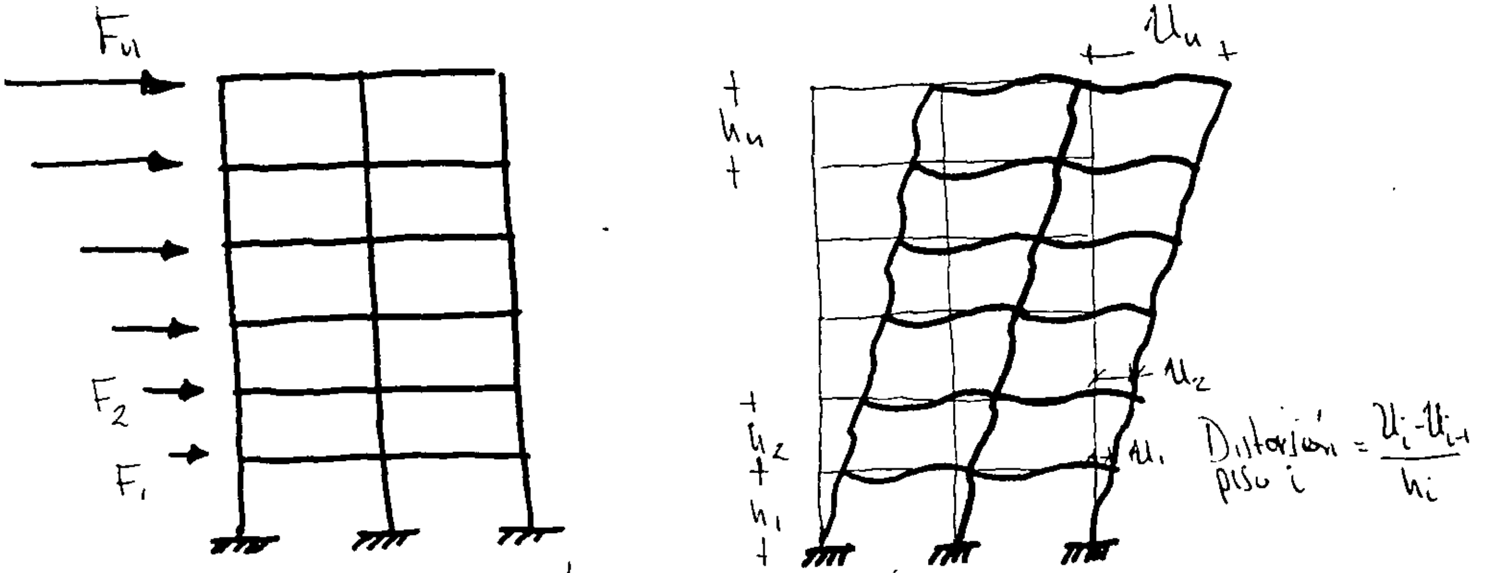


Figura 6 Definición de espectro inelástico de diseño



a) Idealización excitación sísmica

b) Efecto excitación sísmica

Figura 7 Análisis Estructural bajo cargas laterales

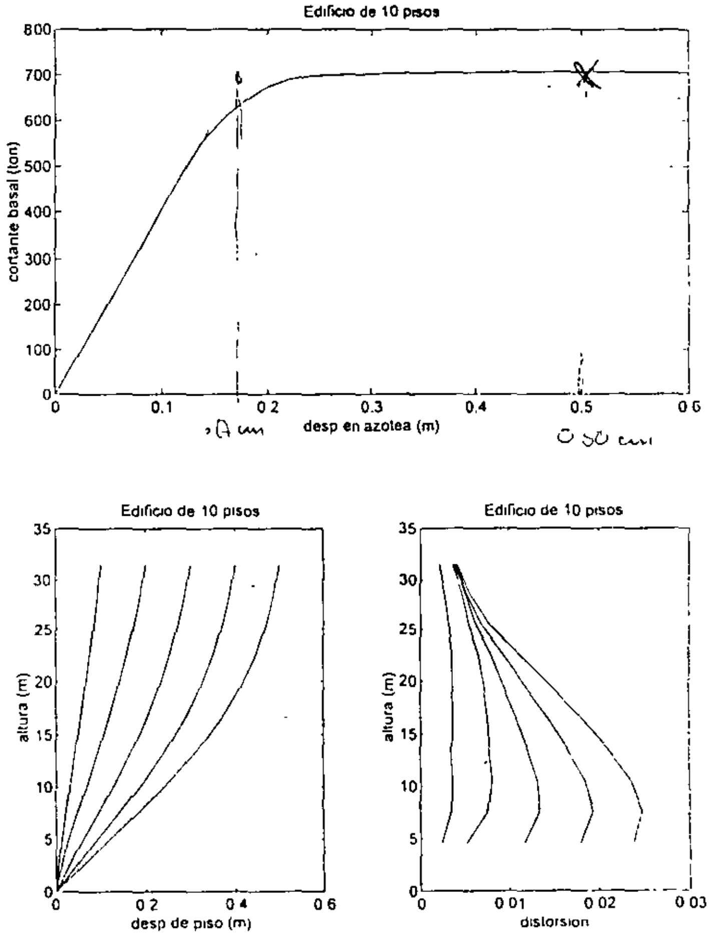


Figura 8 Comportamiento de edificio de 10 pisos

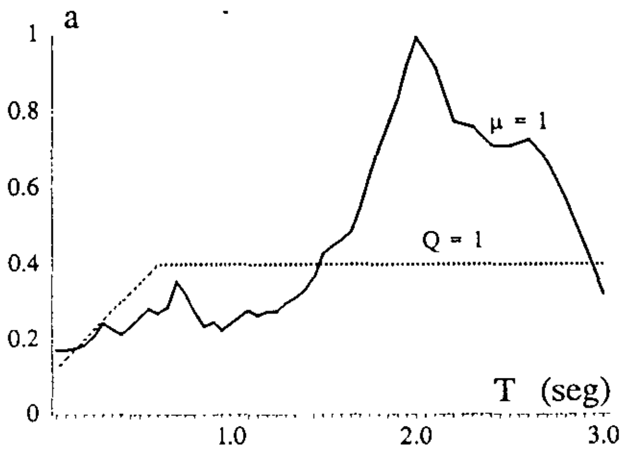


Figura 9 Diferencias entre espectros elásticos de diseño y de respuesta

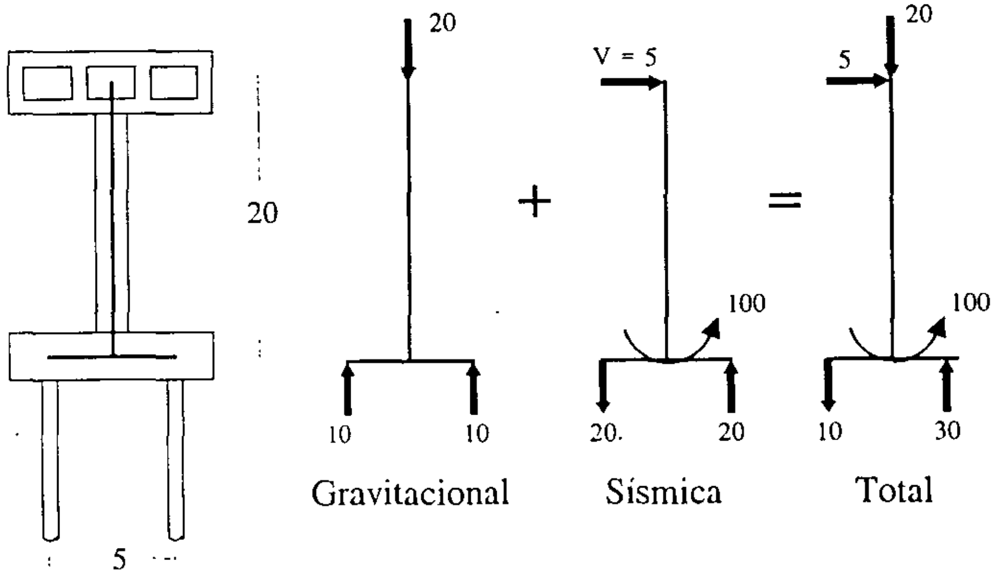
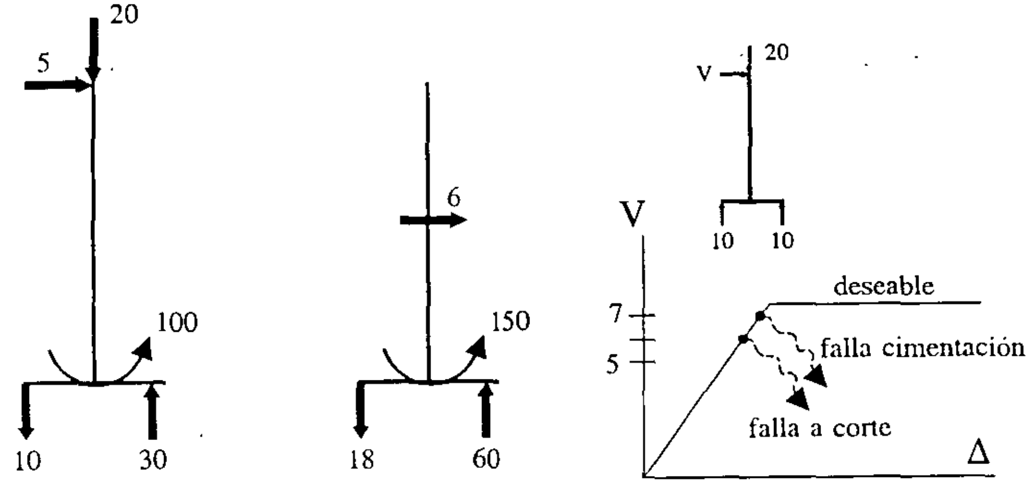


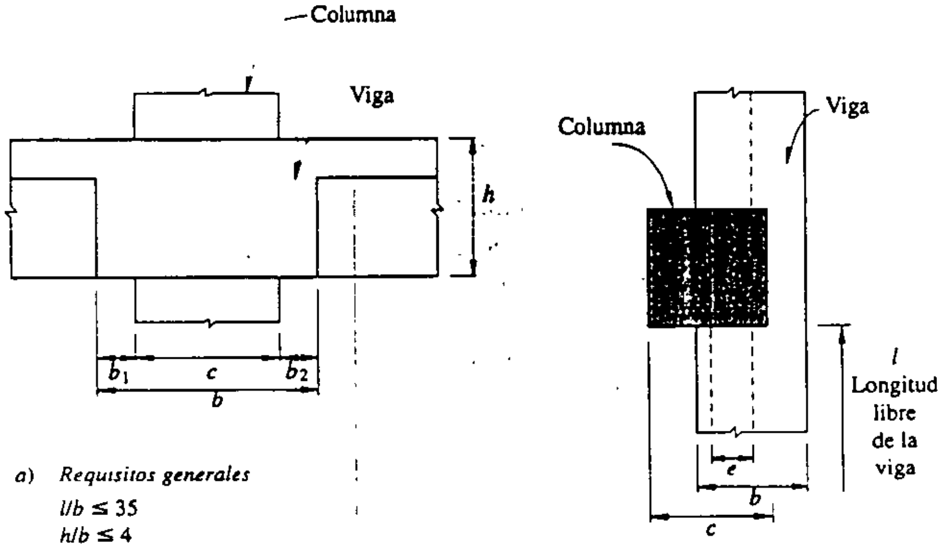
Figura 10 Idealización y acciones de diseño en el paso a desnivel



Demandas

Suministros

Figura 11 Demandas y suministros sísmicos en el paso a desnivel



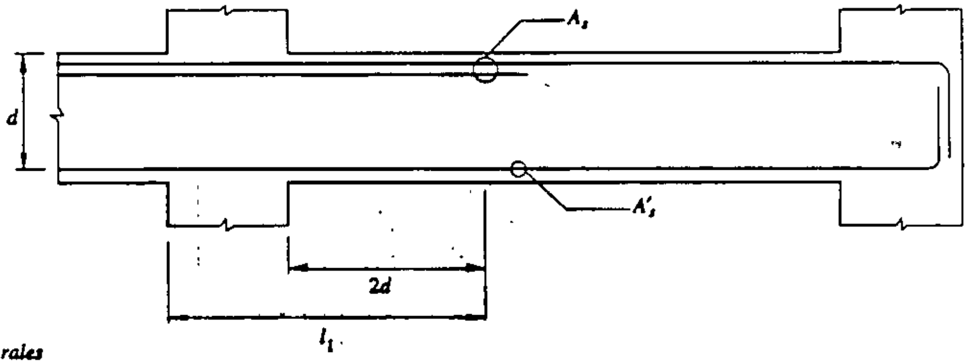
a) *Requisitos generales*

$l/b \leq 35$
$h/b \leq 4$

b) *Requisitos para marcos dúctiles*

$b \geq 25$ *cm* $b_1 = 0$ $h/b \leq 3$
$e/b \leq 0.1$ $l/b \leq 30$ $l/h \geq 4$

Figura 12 Requisitos geométricos para vigas



Requisitos generales

A_s y $A'_s > 0.7 \dfrac{\sqrt{f'_c}}{f_y}$, en zonas donde aparezcan tensiones.

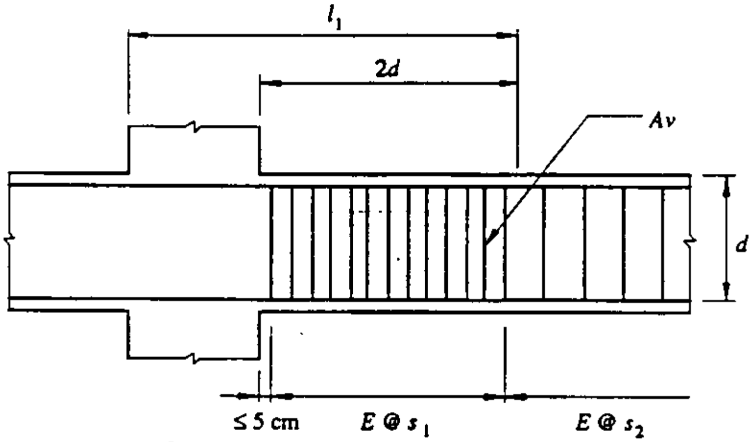
A_s y $A'_s < 0.75\, A_b$ (área de refuerzo correspondiente a la falla balanceada).

Requisitos para marcos dúctiles

A_s y $A'_s \geq 0.7 \dfrac{\sqrt{f'_c}}{f_y}$, en toda la longitud de la viga.

A_s y $A'_s \leq 0.75\, A_{s_b}$.
Mínimo dos barras # 4 en toda la longitud y en ambos lechos.
No se admiten paquetes de más de dos barras.
El momento resistente positivo en l_1 no será menor que la mitad del momento resistente negativo.
No puede haber traslapos, ni corte del refuerzo longitudinal en l_1.
Todo el refuerzo de tensión, A_s, necesario por sismo deberá pasar por el núcleo de la columna.
En toda sección de la viga deberá proporcionarse una resistencia a momento negativo y positivo no menor que una cuarta parte de la máxima que se tiene en los extremos de al viga.

Figura 13 Requisitos para el refuerzo longitudinal de vigas



Requisitos generales

s_1 y $s_2 \leq d/2$ en las zonas donde la fuerza cortante excede de la que resiste el concreto.
Estribos # 2 o mayores.

Requisitos para marcos dúctiles

Estribos # 2.5 o mayores.
En la zona l_1 los estribos deberán ser cerrados y con remate a 135°C, como se indica en la figura 8.4.
La separación no deberá exceder de:

$$s_1 \leq \begin{cases} \text{8 diámetros de la barra longitudinal mayor} \\ \text{24 diámetros del estribo} \\ \text{30 cm} \\ d/4 \end{cases}$$

Además, al menos una de cada dos barras longitudinales de la periferia deberá estar abrazada por la esquina de un estribo.
Fuera de l_1 habrán estribos a una separación $s_2 \leq d/2$.

Figura Requisito para refuerzo transversal de vigas
14

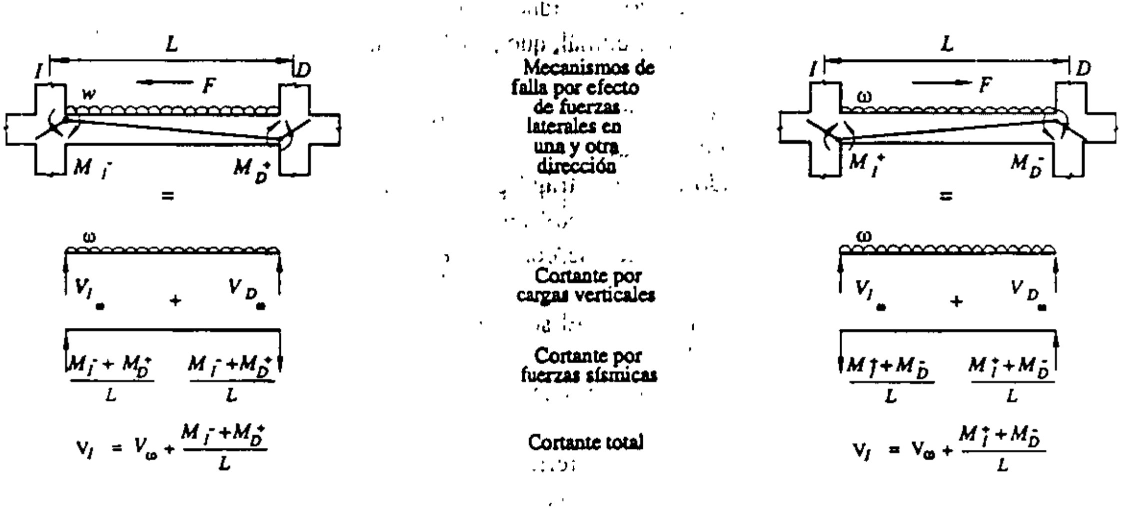
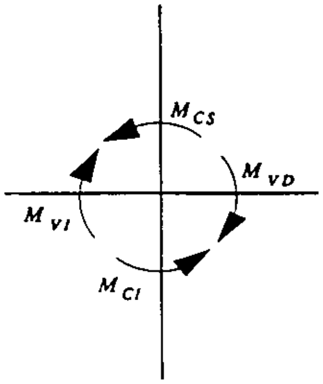


Figura Determinación de fuerzas cortantes de diseño en vigas
15



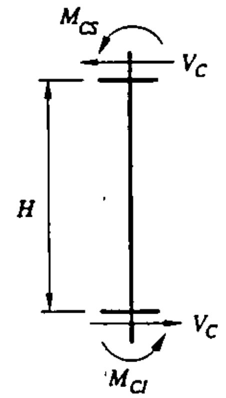
Nudo viga-columna

$\Sigma M_C \geq 1.5\,(\Sigma M_V)$

$\Sigma M_V = M_{VD} + M_{VI}$, es la suma de los momentos flexionantes resistentes (negativo de un lado y positivo del otro) de los extremos de las vigas que llegan a un nudo.

$\Sigma M_C = M_{CS} + M_{CI}$, es la suma de los momentos flexionantes que deben ser capaces de resistir los extremos de las columnas (superior e inferior) que llegan a dicho nudo.

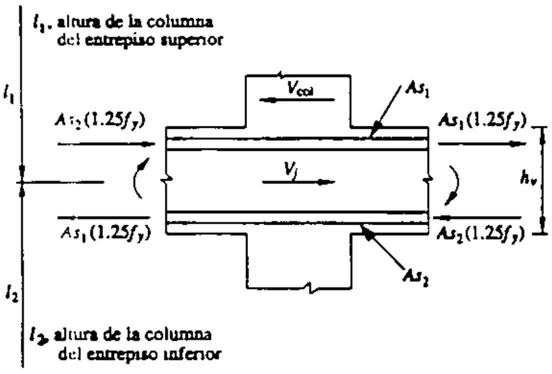
El momento resistente de la columna se calculará para la carga axial que le corresponde a la columna por efecto de la carga vertical más el doble de la que se genera para efecto de las fuerzas sísmicas actuando en la dirección correspondiente al signo de los momentos flexionantes considerados.



$$V_C = \frac{M_{CS} + M_{CI}}{H}$$

$$\Sigma M_C \geq 1.5(\Sigma M_V)$$

Figura 16 Revisión para la capacidad a flexocompresión y cortante de columnas



Por equilibrio del nudo:

$V_j = (As_1 + As_2)1.25\,f_y - V_{col}$

Para la condición de mecanismo de viga se tiene, aproximadamente.

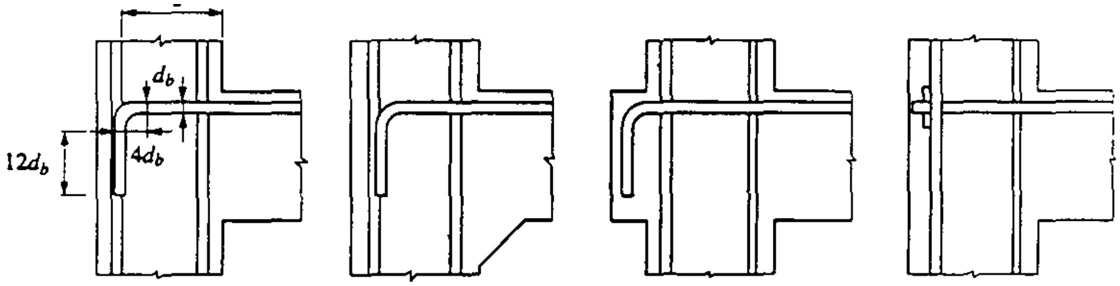
$V_j = (As_1 + As_2)(1.25\,f_y)\left(1 - \frac{1.5\,h_v}{l_1 + l_2}\right)$

No debe excederse de:

$V_j \leq 4.5\,F_R\sqrt{f_c^*}\,b_c\,h_c$

$V_j \leq 5F_R f_c^*\,b_c\,h_c$, cuando hay vigas en las cuatro caras de la unión.

Figura 17 Revisión por cortante de conexiones viga-columna



$$l_a \geq \begin{cases} \dfrac{0.076\, f_y\, d_b}{\sqrt{f'_c}} \\ 15 \text{ cm} \\ 8\, d_b \end{cases}$$

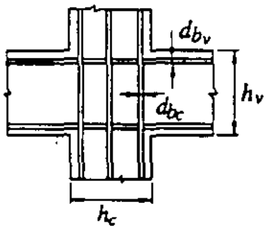
a) Anclaje con gancho extremo.

b) Cartela en extremo de la viga para aumentar la longitud de anclaje con respecto a la sección crítica.

c) Viga con saliente para proporcionar la longitud de anclaje necesaria.

d) Anclaje mecánico.

Figura ~~17~~ 18 Anclaje del refuerzo longitudinal



$$\frac{h_c}{d_{b_v}} \geq 20$$

$\dfrac{h_v}{d_{b_c}} \geq 20$, si la carga axial sobre la columna es tal que:

$$\frac{Pu}{Agf'_c} < 0.3$$

$\dfrac{h_v}{d_{b_c}} \geq 15$, si $\dfrac{Pu}{Agf'_c} \geq 0.3$

Ambos límites se reducen a 15 si más del 50% de las cargas laterales son resistidas por muros o contravientos.

Figura ~~18~~ 19 Requisitos para garantizar adherencia adecuada

FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

XXVI CURSO INTERNACIONAL DE INGENIERIA SÍSMICA

MOUDLO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

TEST SUMMARY N° 1

EXPOSITOR: DR. DAVID DE LEON ESCOBEDO
PALACIO DE MINERIA
SEPTIEMBRE DEL 2000

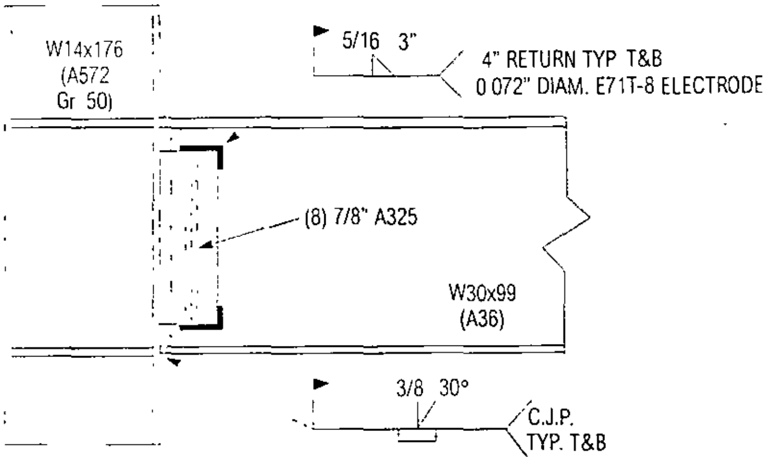
SAC

Test Summary No. 1

the FEMA Program to Reduce the Earthquake Hazards of Steel Moment Frame Structures

Specimen ID.	EERC-PN1
Keywords:	Pre-Northridge, simulated field welding, panel zone yielding, weld fracture, small rotation capacity
Test Location:	Earthquake Engineering Research Center, University of California at Berkeley
Test Date:	March 7, 1995
Principal Investigator:	Vitelmo V. Bertero; with Andrew S. Whittaker and Amir S. Gilani
Related Summaries:	13, 14
Reference:	"Experimental Investigations of Beam-Column Subassemblages," *Report No. SAC 96-01*, March 1996.
Funding Source:	FEMA / SAC Joint Venture, Phase I

CONNECTION DETAIL



MATERIAL PROPERTIES AND SPECIMEN DETAILS

Member	Size	Grade	Yield Stress (ksi)		Ultimate Strength (ksi)	
			mill certs	coupon tests*	mill certs.	coupon tests*
Beam	W30X99	A36	54.1	50.3 flange 55 7 web	73 4	70.9 flange 71 9 web
Column	W14X176	A572 Gr 50	56.5	50 0 flange 49.5 web	74.5	69.0 flange 69.5 web

Welding Procedure Specification	All welds FCAW-SS in conformance with AWSD1.1-94, performed with 0.120" diameter AWS E70T-4 electrodes. Preheat and interpass temperature per Table 4 3. Fillet weld of shear tab to beam web performed with 0.072" diameter AWS E71T-8 electrode
Shear tab	1/2" × 4-1/2" × 23-5/8" plate with eight 7/8" A325 bolts
Panel zone	No doubler plates
Continuity plates	3/8" plates with CJP groove weld
Boundary conditions	Single-sided test, no floor slab, axial load in lower half of column equal to shear in beam, specimen tested in upright position
Other detailing	Connection between column and beam welded in the upright position

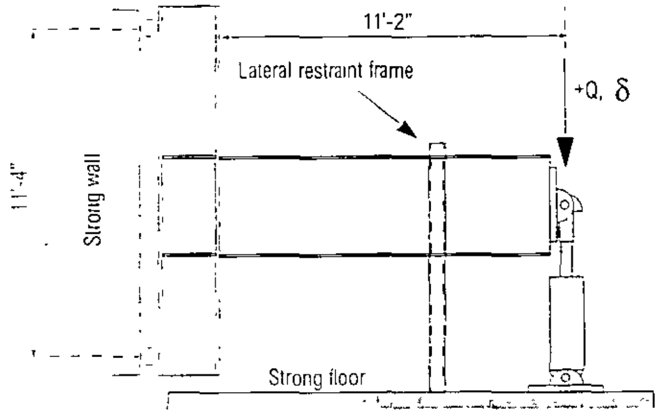
*Coupon locations per ASTM

BACKGROUND

The objectives of testing the Pre-Northridge specimens were to replicate in the laboratory the failure modes observed in the field after the Northridge earthquake to develop a better understanding of the failure mechanisms, and to acquire data on the likely deformation characteristics of beam-column connections constructed to industry standards before 1994. The specimen described in this summary was fabricated under controlled conditions by a local commercial steel fabricator to details specified by SAC and the principal investigator. It was intended to be identical to the specimens described in Test Summaries No. 2 and 3. In addition, these were intended to be nearly identical to the specimens described in Test Summaries No. 4, 5 and 6 which were tested at U.C. San Diego. Because each of these were fabricated under controlled conditions, however, it is possible that their quality is superior to typical moment connections fabricated in the field prior to the Northridge earthquake. As such, some field-fabricated moment connections may exhibit less rotation capacity than these test specimens.

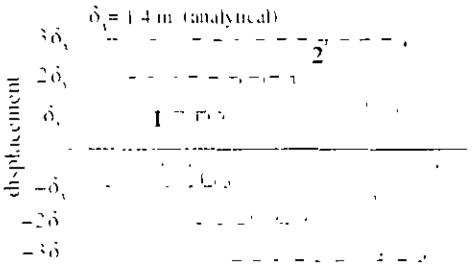
The standard SAC/ATC-24 loading history was used in the quasi-static testing of the specimen. The yield displacement (δ_y) of the specimen was calculated from nonlinear analysis to be 1.40 in.

TEST SET-UP



DISPLACEMENT HISTORY AND KEY EXPERIMENTAL OBSERVATIONS

Applied Displacement History



Key Observations of the Test

Point	Description
1	Shear yielding in the panel zone (displacement = 0.75δ_y)
2	Fracture of welded connection of beam top flange to column flange during first excursion to 3δ_y.

DETAILED TEST RESULTS

Quantity (see Introduction for definitions used in EERC tests)		Maxima
Force/Displacement Properties	Peak actuator force (kips):	110
	Beam deformation (in.)	1.7
	Experimental beam yield displacement (in.)	1.2
Rotation Capacity	Maximum plastic rotation (% radian):	1.1
	Cumulative plastic rotation (% radian):	NA
Energy Dissipation Properties	Cumulative energy dissipated (k-in.):	464

Mode of failure: Fracture of the welded beam top flange to column flange connection during the first half-cycle of loading to 3δ_y

DISCUSSION

Specimen EERC-PN1 failed during the first half-cycle of loading to a displacement of $3\delta_y$. Failure occurred in the groove welded connection of the beam top flange to the column flange at a beam tip displacement of approximately 0.1 in. during the excursion. Failure of the specimen was preceded by shear yielding in the panel zone, initally observed during the first displacement cycle to $0.75\delta_y$. The specimen failed abruptly during the $3\delta_y$ cycle. Data from the strain gages on the top-side of the top flange of the beam indicated flexural strains exceeding 20,000 microstrain. However, visual inspection of the specimen following the testing suggested that there was little plastification over the depth of the beam. The maximum plastic rotation of the connection prior to failure was approximately 1.1% radian: 0.7% radian in the panel zone, and 0.4% radian in the beam. The panel zone dissipated substantially more energy than the beam.

DISCLAIMER

SAC Phase 1 Analytical Studies of Building Performance

Project Title

SAC Survey of Steel Moment-Resisting Frame Buildings Affected by the 1994 Northridge Earthquake

Sub-contractors

David Bonowitz,S.E : Nabih Youssef & Associates
Nabih Youssef, S.E : Nabih Youssef & Associates

Project Summary:

This report presents summaries and preliminary interpretations of data from steel moment-resisting frame buildings shaken by the Northrige earthquake of January, 1994. The subject buildings were investigated for earthquake damage to various degrees at various times since the event. Data was collected for this report between August, 1994 and March, 1995

In general, two patterns of unacceptable conditions are discernible. One pattern is limited to weld root discontinuities that may represent poor construction quality these show little relation to structural demand The other pattern involves full-blown weld or base metal fractures and does appear related to locations of critical structural demand. A remaining question is whether the two patterns are linked, that is, whether poor initial quality leads to fracture during earthquakes. Survey data suggest that poor quality, while it may contribute to damage, is not by itself sufficient to cause damage, even under relatively high stress. Post-Northridge laboratory testing has shown that weld discontinuities are also not necessary for fracture.

Some broad findings regarding general damage patterns and quantities include:

1 About 20% of surveyed buildings reported no damage at all. About 30% of surveyed buildings reported weld damage only.

2 Over 40% of investigated floor-frames were undamaged, and another 40% had weld damage only Among case study buildings, which are somewhat more damaged than average over 70% of individual connections were undamaged, and another 17% had weld damage only. Of all the weld damage reported, about half was limited to root cracks detected by ultrasonic testing.

3 Weld cracks (frequently not confirmed by visual inspection) were reported two to three times as often as base metal fractures. (Cracks in the weld-column fusion zone are generally considered as weld damage.) Weld cracks were reported at the beam bottom flange about three times as often as at the beam top flange. Base metal fractures at the top of the connection were extremely rare. Top inspection was substantially incomplete compared to bottom inspection.

4. Overall, about 16% of floor-frames had some base metal fracture at the bottom of the connection, about 33% had either base metal fracture or significant weld cracks.

5 About 80% of all floor-frames had less than one third of their connections damaged

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The Phase 2 Steel Project has a number of inter-related, problem-focused investigations into various aspects of the steel moment frame problem. These are broken into the following six basic areas:

- Materials and Fracture - evaluating the basic material characteristics of steels used in building construction in seismic regions of the United States
- Joining and Inspection - addressing issues related to welding, bolting, and in-process and post-earthquake inspection
- Connection Performance - beam-column connection modeling and full-scale testing
- System Performance - parametric analytical studies of steel frames with varying ground motions, modeling assumptions, etc.
- Performance Prediction and Evaluation - development of rational and reliable performance-based design procedures for steel moment frames
- Economic, Social, and Political Aspects - identifying opportunities for and barriers against implementing improved seismic design criteria

Performance of Steel Buildings in Past Earthquakes

- Data on Buildings Inspected under the L.A. Inspection Ordinance
- Detailed Data Collection for Selected WSMF Buildings
- Evaluation of Inspection Reliability and Identification of W1a Damage
- Institutional Demands Caused by WSMF Damage
- Damage to Welded Steel Moment Frame Buildings in Earthquakes other than Northridge
- Coordination for Topical Investigations on the Performance of Steel Buildings in Past Earthquakes

Assess Current Knowledge

- Database Development for Sorting and Summarizing Test Results
- Maintenance and Updates of the Literature Database

Materials and Fracture

- Characterization of the Material Properties of Rolled Sections
- Influence of the Through-Thickness Properties upon Beam to Column Flange Welds

Joining and Inspection

- Effects of the Relative Strength of Base and Weld Metal on Weldment Behavior at Different Strain Rates
- Effects of Weld Metal and HAZ Notch Toughness on Welded Joint Behavior
- Sensitivity of Welded Joints to Variations in Welding Procedures, Parameters and Conditions
- Reliability of Non-Destructive Evaluation Methods for Welded Joints - Establish Model UT Procedures
- Synthesize Data Model for Weld Performance
- Establish Weld Acceptance Criteria

Connection Performance

- Assess and Improve Finite Element/Fracture Models for unreinforced

and Cover Plated Connection Details
Assess and Improve Finite Element/Fracture Models for Various Connection Configurations and Types
Develop Analytically-Based Design Methods for Different Connections Configurations
Development and Evaluation of Analytical Tools for Benchmark and Analytical Investigations

System Performance

Develop Suites of Ground Motions
Parametric Study on the Effect of Seismic Demands of Structural Configurations, Proportioning and Modeling
Parametric Study on the Effect of Seismic Demands of Deterioration of Hysteretic Characteristics
Parametric Study on the Effects on Seismic Demands of Ground Motion Intensity and Dynamic Characteristics
Investigation of the Effect of Connection Fractures on Safety and Reliability of Steel Moment Resisting Frame Systems
Investigation of Alternative Framing Systems

Performance, Prediction and Evaluation

Develop a Statistical and Reliability Framework for Comparing and Evaluation Predictive Models for Evaluation and Design
Develop Predictive Elastic Models for Building Performance
Develop Predictive Inelastic Models for Building Performance
Develop Recommended Procedures for Performance and Evaluation of SMRF Buildings
Develop Design Requirements for Ordinary and Intermediate (Low Ductility) Moment Frames

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FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

XXVI CURSO INTERNACIONAL DE INGENIERIA SÍSMICA

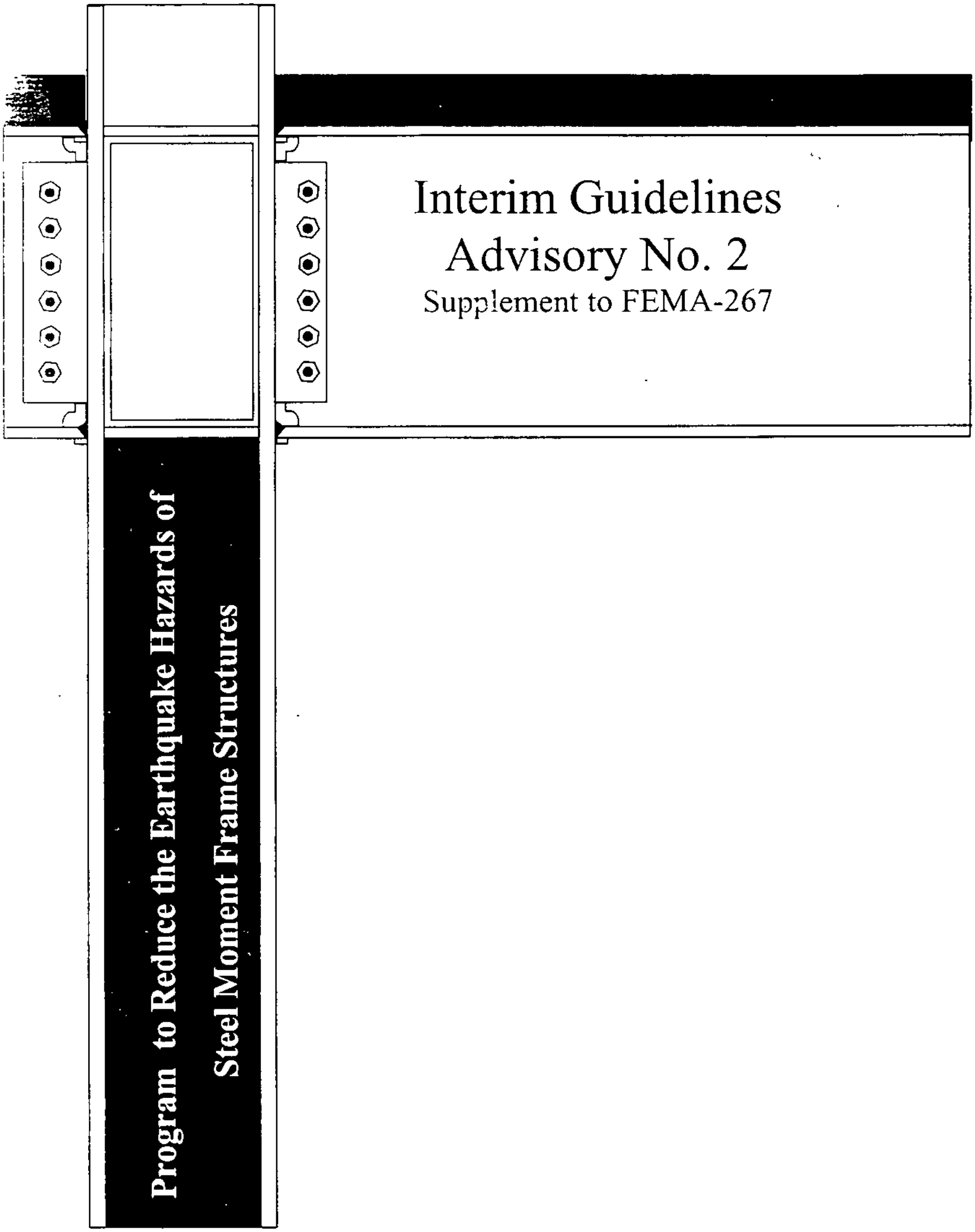
MOUDLO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

INTERIM GUIDELINES ADVISORY N° 2

EXPOSITOR: DR. DAVID DE LEON ESCOBEDO
PALACIO DE MINERIA
SEPTIEMBRE DEL 2000

FEDERAL EMERGENCY MANAGEMENT AGENCY
FEMA 267b / June, 1999



Interim Guidelines
Advisory No. 2

Supplement to FEMA-267

Program to Reduce the Earthquake Hazards of
Steel Moment Frame Structures

INTERIM GUIDELINES ADVISORY NO. 2

Supplement to FEMA-267 Interim Guidelines:

Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures

Report No. SAC-99-01

SAC Joint Venture

a partnership of:
Structural Engineers Association of California (SEAOC)
Applied Technology Council (ATC)
California Universities for Research in Earthquake Engineering (CUREe)

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June, 1999

THE SAC JOINT VENTURE

SAC is a joint venture of the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and California Universities for Research in Earthquake Engineering (CUREe,) formed specifically to address both immediate and long-term needs related to solving problems of the Welded Steel Moment Frame (WSMF) connection that became apparent as a result of the 1994 Northridge earthquake. SEAOC is a professional organization composed of more than 3,000 practicing structural engineers in California The volunteer efforts of SEAOC's members on various technical committees have been instrumental in the development of the earthquake design provisions contained in the *Uniform Building Code* as well as the *National Earthquake Hazards Reduction Program (NEHRP) Provisions for Seismic Regulations for New Buildings*. The Applied Technology Council is a non-profit organization founded specifically to perform problem-focused research related to structural engineering and to bridge the gap between civil engineering research and engineering practice. It has developed a number of publications of national significance including ATC 3-06, which serves as the basis for the *NEHRP Recommended Provisions.* CUREe is a nonprofit organization formed to promote and conduct research and educational activities related to earthquake hazard mitigation. CUREe's eight institutional members are: the California Institute of Technology, Stanford University, the University of California at Berkeley, the University of California at Davis, the University of California at Irvine, the University of California at Los Angeles, the University of California at San Diego, and the University of Southern California This collection of university earthquake research laboratory, library, computer and faculty resources is among the most extensive in the United States. The SAC Joint Venture allows these three organizations to combine their extensive and unique resources, augmented by subcontractor universities and organizations from around the nation, into an integrated team of practitioners and researchers, uniquely qualified to solve problems related to the seismic performance of WSMF structures

DISCLAIMER

The purpose of this document is to serve as a supplement to the *FEMA-267* publication *Interim Guidelines. Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures* . This Advisory, which is intended to be used in conjunction with *FEMA-267*, supercedes and entirely replaces *Interim Guidelines Advisory No. 1* (*FEMA 267a*). *FEMA-267* was published to provide engineers and building officials with guidance on engineering procedures for evaluation, repair, modification and design of welded steel moment frame structures, to reduce the risks associated with earthquake-induced damage The recommendations were developed by practicing engineers based on professional judgment and experience and a preliminary program of laboratory, field and analytical research. This preliminary research, known as the SAC Phase 1 program, commenced in November, 1994 and continued through the publication of the *Interim Guidelines* document. This *Interim Guidelines Advisory No. 2*, which updates and replaces *Interim Guidelines Advisory No 1*, is based on supplementary data developed under a program of continuing research, known as the SAC Phase 2 program, as well as findings developed by other, independent researchers. Final design recommendations, superceeding both *FEMA-267* and this document are scheduled for publication in early 2000 Independent review and guidance in the production of both the *FEMA-267*, *Interim Guidelines* and the advisories was provided by a project oversight panel comprised of experts from industry, practice and academia. Users are cautioned that research into the behavior of these structures is continuing. Interpretation of the results of this research may invalidate or suggest the need for modification of recommendations contained herein. **No warranty is offered with regard to the recommendations contained herein, either by the Federal Emergency Management Agency, the SAC Joint Venture, the individual joint venture partners, their directors, members or employees. These organizations and their employees do not assume any legal liability or responsibility for the accuracy, completeness, or usefulness of any of the information, products or processes included in this publication. The reader is cautioned to carefully review the material presented herein.** Such information must be used together with sound engineering judgment when applied to specific engineering projects This *Interim Guidelines Advisory* has been prepared by the SAC Joint Venture with funding provided by the Federal Emergency Management Agency, under contract number EMW-95-C-4770 The SAC Joint Venture gratefully acknowledges the support of FEMA and the leadership of Michael Mahoney and Robert Hanson Project Officer and Technical Advisor, respectively. The SAC Joint Venture also wishes to express its gratitude to the large numbers of engineers, building officials, organizations and firms that provided substantial efforts, materials, and advice and who have contributed significantly to the progress of the Phase 2 effort.

PREFACE

Purpose

The purpose of the *Interim Guidelines Advisory* series is to provide engineers and building officials with timely information and guidance resulting from ongoing problem-focused studies of the seismic behavior of moment-resisting steel frame structures. These advisories are intended to be supplements to *FEMA-267 Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures* first published in August 1995.

The first *Interim Guidelines Advisory, FEMA-267a,* was published in January 1997. The specific revisions and updates to the *Interim Guidelines* contained in *FEMA-267a* were developed based on input obtained from a group of engineers and building officials actively engaged in the use of the *FEMA-267* document, in the period since its initial publication in August 1995. That input was obtained during a workshop held in August 1996, in Los Angeles, California.

This second Interim Guidelines Advisory has been prepared as a series of updates and revisions both to the *FEMA-267, Interim Guidelines* which it supplements and to the *FEMA-267a, Interim Guidelines Advisory* publication, which it supercedes. The material contained in this *Interim Guidelines Advisory No. 2* is based on the extensive analytical and laboratory research that has been conducted by the SAC Joint Venture and other researchers during the intervening period, along with recent developments in the steel construction industry. The material contained in this *Advisory* has been formatted to match that contained in the original *Interim Guidelines*, to permit the user to insert this material directly into appropriate sections of that document. This *Advisory* is not intended to serve as a self-contained text and should not be used as such. It does, however, completely replace the material contained in *FEMA-267a.*

A new set of recommendations for the design, analysis, evaluation repair, retrofit and construction of moment-resisting steel frames is currently being prepared as part of the Phase 2 Program to Reduce Earthquake Hazards in Steel Moment Frame Structures. These new Seismic Design Criteria, which are anticipated to be completed early in the year 2000, will replace in their entirety the *FEMA-267 Interim Guidelines* and this *Interim Guidelines Advisory No. 2.*

Background

The Northridge earthquake of January 17, 1994, dramatically demonstrated that the prequalified, welded beam-to-column moment connection commonly used in the construction of welded steel moment resisting frames (WSMFs) in the period 1965-1994 was much more susceptible to damage than previously thought. The stability of moment frame structures in earthquakes is dependent on the capacity of the beam-column connection to remain intact and to resist tendencies of the beams and columns to rotate with respect to each other under the influence of lateral deflection of the structure. The prequalified connections were believed to be ductile and capable of withstanding the repeated cycles of large inelastic deformation explicitly relied upon in the building code provisions for the design of these structures. Although many affected connections were not damaged, a wide spectrum of unexpected brittle connection fractures did occur, ranging from isolated fractures through or adjacent to the welds of beam flanges to columns, to large fractures extending across the full depth of the columns. At the time this damage was discovered, the structural steel industry and engineering profession had little understanding of the specific causes of this damage, the implications of this damage for building safety, or even if reliable methods existed to repair the damage which had been discovered. Although the connection failures did not result in any casualties or collapses, and many WSMF buildings were not damaged, the incidence of damage was sufficiently pervasive in regions of strong ground motion to cause wide-spread concern by structural engineers and building officials with regard to the safety of these structures in future earthquakes.

In response to these concerns, the Federal Emergency Management Agency (FEMA) entered into a cooperative agreement with the SAC Joint Venture to perform problem-focused study of the seismic performance of welded steel moment connections and to develop interim recommendations for professional practice. Specifically, these recommendations were intended to address the inspection of earthquake affected buildings to determine if they had sustained significant damage, the repair of damaged buildings; the upgrade of existing buildings to improve their probable future performance; and the design of new structures to provide more reliable seismic performance Within weeks of receipt of notification of FEMA's intent to enter into this agreement, the SAC Joint Venture published a series of two design advisories (SAC, 1994a; SAC, 1994b) These design advisories presented a series of papers, prepared by engineers and researchers engaged in the investigation of the damaged structures and presenting individual opinions as to the causes of the damage, potential methods of repair, and possible designs for more reliable connections in the future. In February 1995, *Design Advisory No. 3* (SAC, 1995a) was published. This third advisory presented a synthesis of the data presented in the earlier publications, together with the preliminary recommendations developed in an industry workshop, attended by more than 50 practicing engineers, industry representatives and researchers, on methods of inspecting, repairing and designing WSMF structures. At the time this third advisory was published, significant disagreement remained within the industry and the profession as to the specific causes of the damage observed and appropriate methods of repair given that the damage had occurred Consequently, the preliminary recommendations were presented as a series of issue statements, followed by the consensus opinions of the workshop attendees, where consensus existed, and by majority and dissenting opinions where such consensus could not be formed.

During the first half of 1995, an intensive program of research was conducted to more definitively explore the pertinent issues. This research included literature surveys, data collection on affected structures, statistical evaluation of the collected data, analytical studies of damaged and undamaged buildings and laboratory testing of a series of full-scale beam-column assemblies representing typical pre-Northridge design and construction practice as well as various repair, upgrade and alternative design details The findings of this research (SAC 1995c, SAC 1995d, SAC 1995e, SAC 1995f, SAC 1995g, SAC 1996) formed the basis for the development of FEMA 267 - *Interim Guidelines: Evaluation, Repair, Modification, and Design of Welded Steel Moment Frame Structures* (SAC, 1995b), which was published in August, 1995. FEMA 267 provided the first definite, albeit interim, recommendations for practice, following the discovery of connection damage in the Northridge earthquake.

As a result of these and supplemental studies conducted by the SAC Joint Venture, as well as independent research conducted by others, it is now known that a large number of factors contributed to the damage sustained by steel frame buildings in the Northridge earthquake. These included:

- design practice that favored the use of relatively few frame bays to resist lateral seismic demands, resulting in much larger member and connection geometries than had previously been tested,
- standard detailing practice which resulted in the development of large inelastic demands at the beam to column connections;
- detailing practice that often resulted in large stress concentrations in the beam-column connection, as well as inherent stress risers and notches in zones of high stress;
- the common use of welding procedures that resulted in deposition of low toughness weld metal in the critical beam flange to column flange joints;
- relatively poor levels of quality control and assurance in the construction process, resulting in welded joints that did not conform to the applicable quality standards;
- excessively weak and flexible column panel zones that resulted in large secondary stresses in the beam flange to column flange joints;
- large variations in the strengths of rolled shape members relative to specified values;
- an inherent inability of material to yield under conditions of high tri-axial restraint such as exist at the center of the beam flange to column flange joints.

With the identification of these factors it was possible for *FEMA 267* to present a recommended methodology for the design and construction of moment-resisting steel frames to provide connections capable of more reliable seismic performance. This methodology included the following recommendations

- proportion the beam-column connection such that inelastic behavior occurs at a distance remote from the column face, minimizing demands on the highly restrained column material and the welded joints;
- specify weld filler metals with rated toughness values for critical welded joints;
- detail connections to incorporate beam flange continuity plates, to minimize stress concentrations;
- remove backing bars and weld tabs from critical joints to minimize the potential for stress risers and notch effects and also to improve the reliability with which flaws at the weld root can be observed and repaired;

- qualify connection configurations through a program of full-scale inelastic testing of representative beam-column assemblies, fabricated in the same manner as is proposed for use in the structure;

- increased participation of the design professional in the specification and surveillance of welding procedures and the quality assurance process for welded joints.

In the time since the publication of *FEMA-267*, SAC has continued, under funding provided by FEMA, to perform problem-focused study of the performance of moment resisting connections of various configurations. This work, which is generally referred to as the SAC Phase II program, includes detailed analytical evaluations of buildings and connections, parametric studies into the effects on connection performance of connection configuration, base and weld metal strength, toughness and ductility, as well as additional large scale testing of connection assemblies. The intent of this study is to support development of final guidelines that will present more reliable and economical performance-based methods for:

- identification of damaged structures following an earthquake and determination of the extent, severity and consequences of such damage;

- design of effective repairs for damaged structures;

- identification of existing structures that are vulnerable to unacceptable levels of damage in future earthquakes;

- design of structural upgrades for existing vulnerable structures;

- design of new structures that are suitably resistant to earthquake induced damage;

- procedures for construction quality assurance that are consistent with the levels of reliability intended by the design criteria.

This Phase II program of research, which is being conducted by the SAC Joint Venture in parallel and coordination with work by other researchers, is anticipated to be complete in late 1999. It is the intent of FEMA and the SAC Joint Venture to ensure that pertinent information and findings from this program are made available to the user community in a timely manner through the publication of this series of design advisory documents. This *Interim Guidelines Advisory No. 2* is the second such publication.

Format

This *Advisory* has been prepared as a series of updates and revisions to the *FEMA-267, Interim Guidelines* publication. It has been formatted in a manner intended to facilitate the identification of changes to the original *FEMA-267* text. Only those sections of *FEMA-267* that are being revised at this time are included. Other sections of *FEMA-267* remain in effect as the current best recommendations of the SAC Joint Venture. This *Advisory* replaces the earlier *Interim Guidelines Advisory, FEMA-267a*, in its entirety.

To facilitate coordination of this *Advisory* with *FEMA-267*, the existing system of chapter and section numbering has been retained. The Table of Contents lists all sections of the chapters being revised, including those sections for which no revisions are included. Within the body of this document, a section heading is provided for each section of the chapter; however, if no revision to the section is currently being made, this is indicated immediately beneath the section heading.

To facilitate reading of this document, where a revision is made to a section in *FEMA 267*, the entire text of that section is included herein. Where existing text from *FEMA-267* is reproduced in this document, without edit, it is shown in normal face type for guidelines, and in italicized type for commentary. Where existing text is being deleted, this is shown in strike through format. A single strikethrough indicates text deleted in the first advisory, *FEMA-267a*. A double strikethrough indicates text deleted in this current advisory. New text is shown in underline format. A single underline identifies text added in the first advisory, *FEMA-267a*. A double underline identifies text added in this current advisory. When a modification has been made to a portion of text, relative to *FEMA-267*, this will also be noted by the presence of a vertical line at the outside margin of the page. The following two paragraphs illustrate these conventions for guideline and commentary text, respectively.

> This sentence is representative of typical guideline text, that has been reprinted from FEMA-267 without change.~~This sentence, is representative of the way in which text being deleted from FEMA-267 in this *Interim Guidelines Advisory* is identified.~~ ~~This sentence illustrates the way in which text deleted from FEMA-267 in the previous *Interim Guidelines Advisory* is identified.~~ This sentence illustrates the way in which text being added to *FEMA-267* in this *Interim Guidelines Advisory* is identified.This sentence illustrates the way in which text added to FEMA-267 in the previous *Interim Guidelines Advisory* is identified.

> *Commentary: This sentence is representative of typical commentary text, that has been reprinted from FEMA-267 without change.* ~~*This sentence is representative of the way in which commentary text being deleted from FEMA-267 in this Interim Guidelines Advisory is identified.*~~ ~~*However, this sentence, is representative of the way in which text being deleted from FEMA-267 commentary in the previous advisory is identified.*~~ *This sentence indicates the way in which text added to the FEMA-267 commentary in this Advisory is shown.This final sentence illustrates the way in which text added in previous advisory FEMA-267a, is identified.*

Intent

This *Interim Guidelines Advisory*, together with the *Interim Guidelines* they modify, are primarily intended for two different groups of potential users:

a) Engineers engaged in evaluation, repair, and upgrade of existing WSMF buildings and in the design of new WSMF buildings incorporating either Special Moment-Resisting Frames or Ordinary Moment-Resisting Frames utilizing welded beam-column connections. The recommendations for new construction are applicable to all WSMF construction expected to resist earthquake demands through plastic behavior.

b) Regulators and building departments responsible for control of the evaluation, repair, and occupancy of WSMF buildings that have been subjected to strong ground motion and for regulation of the design, construction, and inspection of new WSMF buildings.

The fundamental goal of the information presented in the *Interim Guidelines* as modified by this *Advisory* is to help identify and reduce the risks associated with earthquake-induced fractures in WSMF buildings through provision of timely information on how to inspect existing buildings for damage, repair damage if found, upgrade existing buildings and design new buildings. The information presented here primarily addresses the issue of beam-to-column connection integrity under the severe inelastic demands that can be produced by building response to strong ground motion. Users are referred to the applicable provisions of the locally prevailing building code for information with regard to other aspects of building construction and earthquake damage control.

Limitations

The information presented in this *Interim Guidelines Advisory*, together with that contained in the *Interim Guidelines* it modifies, is based on limited research conducted since the Northridge Earthquake, review of past research and the considerable experience and judgment of the professionals engaged by SAC to prepare and review this document. Additional research on such topics as the effect of floor slabs on frame behavior, the effect of weld metal and base metal toughness, the efficacy of various beam-column connection details and the validity of current standard testing protocols for prediction of earthquake performance of structures is continuing as part of the Phase 2 program and is expected to provide important information not available at the time this *Advisory* was formulated. Therefore, many of the recommendations cited herein may change as a result of forthcoming research results

The recommendations presented herein represent the group consensus of the committee of Guideline Writers retained by SAC following independent review by the Project Oversight Committee They may not reflect the individual opinions of any single participant. They do not necessarily represent the opinions of the SAC Joint Venture, the Joint Venture partners, or the sponsoring agencies. Users are cautioned that available information on the nature of the WSMF problem is in a rapid stage of development and any information presented herein must be used with caution and sound engineering judgment.

TABLE OF CONTENTS

1. INTRODUCTION

1.1 Purpose

There are no modifications to the Guidelines or Commentary of Section 1.1 at this time.

1.2 Scope

There are no modifications to the Guidelines or Commentary of Section 1.2 at this time.

1.3 Background

Following the January 17, 1994 Northridge, California Earthquake, more than 100 steel buildings with welded moment-resisting frames were found to have experienced beam-to-column connection fractures. The damaged structures cover a wide range of heights ranging from one story to 26 stories; and a wide range of ages spanning from buildings as old as 30 years of age to structures just being erected at the time of the earthquake. The damaged structures ~~are~~ were spread over a large geographical area, including sites that experienced only moderate levels of ground shaking. Although relatively few such buildings were located on sites that experienced the strongest ground shaking, damage to these buildings was quite severe. Discovery of these extensive connection fractures, often with little associated architectural damage to the buildings,was ~~has been~~ alarming. The discovery~~has~~ also caused some concern that similar, but undiscovered damage may have occurred in other buildings affected by past earthquakes. Indeed, there are now confirmed ~~isolated~~ reports of such damage. In particular, a publicly owned building at Big Bear Lake is~~known to have been~~ was damaged by the Landers-Big Bear, California sequence of earthquakes.~~and at least one building, under construction in Oakland, California at the timefo the~~ several buildings were damaged during the1989 Loma Prieta Earthquake~~, was reported to have experienced such damage~~ in the San Francisco Bay Area

WSMF construction is used commonly throughout the United States and the world, particularly for mid- and high-rise construction. Prior to the Northridge Earthquake, this type of construction was considered one of the most seismic-resistant structural systems, due to the fact that severe damage to such structures had rarely been reported in past earthquakes and there was no record of earthquake-induced collapse of such buildings, constructed in accordance with contemporary US practice. However, the widespread severe structural damage which occurred to such structures in the Northridge Earthquake called~~s~~ for re-examination of this premise.

The basic intent of the earthquake resistive design provisions contained in the building codes is to protect the public safety, however, there is also an intent to control damage. The developers of the building code provisions have explicitly set forth three specific performance goals for buildings designed and constructed to the code provisions (SEAOC - 1990). These are to provide buildings with the capacity to

- resist minor earthquake ground motion without damage;

- resist moderate earthquake ground motion without structural damage but possibly some nonstructural damage, and

- resist major levels of earthquake ground motion, having an intensity equal to the strongest either experienced or forecast for the building site, without collapse, but possibly with some structural as well as nonstructural damage.

In general, WSMF buildings in the Northridge Earthquake met the basic intent of the building codes, to protect life safety. However, the ground shaking intensity experienced by most of these buildings was significantly less than that anticipated by the building code. Many buildings that experienced moderate intensity ground shaking experienced significant damage that could be viewed as failing to meet the intended performance goals with respect to damage control. Further, some members of the engineering profession (SEAOC - 1995b) and government agencies (Seismic Safety Commission - 1995) have stated that even these performance goals are inadequate for society's current needs.

WSMF buildings are designed to resist earthquake ground shaking based on the assumption that they are capable of extensive yielding and plastic deformation, without loss of strength. The intended plastic deformation is intended to be developed through a combination of ~~consists of~~ plastic rotations developing within the beams, at their connections to the columns and plastic shear yielding of the column panel zones. ~~and is t~~Theoretically these mechanisms should be capable of resulting in benign dissipation of the earthquake energy delivered to the building. Damage is expected to consist of moderate yielding and localized buckling of the steel elements, not brittle fractures. Based on this presumed behavior, building codes require a minimum lateral design strength for WSMF structures that is approximately 1/8 that which would be required for the structure to remain fully elastic. Supplemental provisions within the building code, intended to control the amount of interstory drift sustained by these flexible frame buildings, typically result in structures which are substantially stronger than this minimum requirement and in zones of moderate seismicity, substantial overstrength may be present to accommodate wind and gravity load design conditions. In zones of high seismicity, most such structures designed to minimum code criteria will not start to exhibit plastic behavior until ground motions are experienced that are 1/3 to 1/2 the severity anticipated as a design basis. This design approach has been developed based on historical precedent, the observation of steel building performance in past earthquakes, and limited research that has included laboratory testing of beam-column models, albeit with mixed results, and non-linear analytical studies.

Observation of damage sustained by buildings in the Northridge Earthquake indicates that contrary to the intended behavior, in some ~~many~~ cases brittle fractures initiated within the connections at very low levels of plastic demand, and in some cases, while the structures remained essentially elastic. Typically, but not always, fractures initiated at, or near, the complete joint penetration (CJP) weld between the beam bottom flange and column flange (Figure 1-1). Once initiated, these fractures progressed along a number of different paths, depending on the individual joint and stress conditions. Figure 1-1 indicates just one of these potential fracture growth patterns. ~~Investigators initially identified a number of factors which may have contributed to the initiation of fractures at the weld root including~~: ~~notch effect, created by the backing bar which was commonly left in place following joint completion~~; ~~sub-standard welding that included excessive porosity and slag inclusions as well as incomplete fusion~~;

~~and potentially, pre-earthquake fractures resulting from initial shrinkage of the highly restrained weld during cool-down. Such problems could be minimized in future construction, with the application of appropriate welding procedures and more careful exercise of quality control during the construction process. However, it is now known that these were not the only causes of the fractures which occurred.~~

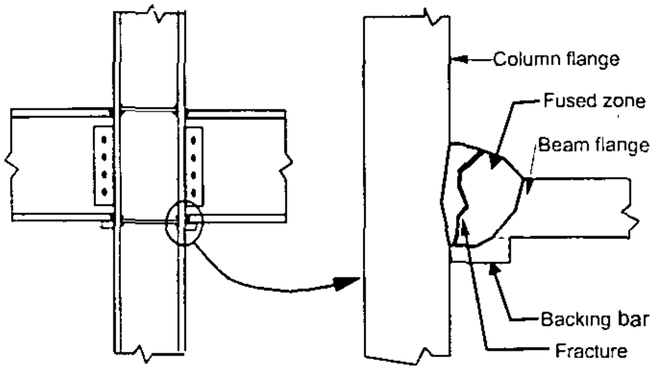


Figure 1-1 - Common Zone of Fracture Initiation in Beam -Column Connection

~~Current production processes for structural steel shapes result in inconsistent strength and deformation capacities for the material in the through-thickness direction. Non-metallic inclusions in the material, together with anisotropic properties introduced by the rolling process can lead to lamellar weakness in the material. Further, the distribution of stress across the girder flange, at the connection to the column is not uniform. Even in connections stiffened by continuity plates across the panel zone, significantly higher stresses tend to occur at the center of the flange, where the column web produces a local stiffness concentration. Large secondary stresses are also induced into the girder flange to column flange joint by kinking of the column flanges resulting from shear deformation of the column panel zone.~~

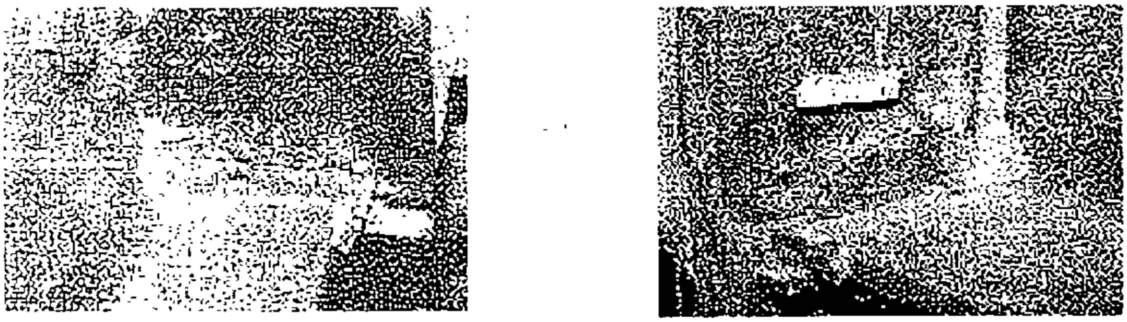
~~The dynamic loading experienced by the moment-resisting connections in earthquakes~~ is ~~characterized by high-strain tension-compression cycling. Bridge engineers have long recognized that the dynamic loading associated with bridges necessitates different connection details in order to provide improved fatigue resistance, as compared to traditional building design that is subject to "static" loading due to gravity and wind loads. While the nature of the dynamic loads resulting from earthquakes is somewhat different than the high cycle dynamic loads for which fatigue prone structures are designed, similar detailing may be desirable for buildings subject to seismic loading~~.

~~In design and construction practice for welded steel bridges, mechanical and metallurgical notches should be avoided because they may be the initiators of fatigue cracking. As fatigue cracks grow under repetitive loading, a critical crack size may be reached whereupon the material toughness (which is~~ a ~~function of temperature) may be unable to resist the onset of brittle (unstable) crack growth. The beam-to-column connections in WSMF buildings are comparable to category C or D bridge details that have a reduced allowable stress range as opposed to category B details for which special metallurgical~~, ~~inspection and testing requirements are applied. The rapid rate of loading imposed by seismic events~~, ~~and the complete inelastic range of tension-compression-tension loading applied to these connections~~ is

~~much more severe than typical bridge loading applications. The mechanical and metallurgical notche~~s ~~or stress risers created by the beam-column weld joints are a logical point for fracture problems t~~o ~~initiate. This, coupled with the tri-axial restraint provided by the beam web and the column flange, is~~ a ~~recipe for brittle fracture~~.

~~During the Northridge Earthquake, o~~Once fractures initiated in beam-column joints, they progressed in a number of different ways. ~~In some cases, the fractures initiated but did not grow, and could not be detected by visual observation. In other cases,~~ In many cases, the fractures progressed ~~completely~~ directly through the thickness of the weld, and if fireproofing was removed, the fractures were evident as a crack through exposed faces of the weld, or the metal just behind the weld (Figure 1-2a). Other fracture patterns also developed. In some cases, the fracture developed into a surface that resembled a through-thickness failure of the column flange material behind the CJP weld (Figure 1-2b). In these cases, a portion of the column flange remained bonded to the beam flange, but pulled free from the remainder of the column. This fracture pattern has sometimes been termed a "divot" or "nugget" failure.

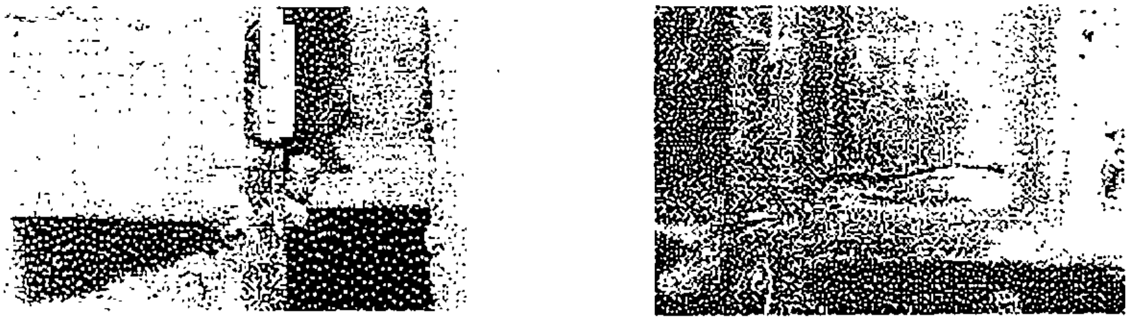
A number of fractures progressed completely through the column flange, along a near horizontal plane that aligns approximately with the beam lower flange (Figure 1-3a). In some cases, these fractures extended into the column web and progressed across the panel zone Figure (1-3b). Investigators have reported some instances where columns fractured entirely across the section.



a. Fracture at Fused Zone

b. Column Flange "Divot" Fracture

Figure 1-2 - Fractures of Beam to Column Joints



a Fractures through Column Flange

b. Fracture Progresses into Column Web

Figure 1-3 - Column Fractures

Once these fractures have occurred, the beam - column connection has experienced a significant loss of flexural rigidity and capacity. Residual flexural strength and rigidity must be developed through a couple consisting of forces transmitted through the remaining top flange connection and the web bolts. ~~Initial~~ Research suggests that residual stiffness is approximately 20% of that of the undamaged connection and that residual strength varies from 10% to 40% of the undamaged capacity, when loading results in tensile stress normal to the fracture plane. When loading produces compression across the fracture plane, much of the original strength and stiffness remain. However, in providing this residual strength and stiffness, the beam shear connections can themselves be subject to failures, consisting of fracturing of the welds of the shear plate to the column, fracturing of supplemental welds to the beam web or fracturing through the weak section of shear plate aligning with the bolt holes (Figure 1-4).

Figure 1-4 - Vertical Fracture through Beam Shear Plate Connection

It is now known that these fractures were the result of a number of complex factors that were not well understood either when these connections were first adopted as a standard design approach, or when the damage was discovered immediately following the Northridge earthquake. Engineers had commonly assumed that when these connections were loaded to yield levels, flexural stresses in the beam would be transferred to the column through a force couple comprised of nearly uniform yield level tensile and compressive stresses in the beam flanges. It was similarly assumed that nearly all of the shear stress in the beam was transferred to the column through the shear tab connection to the beam web. In fact, the actual behavior is quite different from this. As a result of local deformations that occur in the column at the location of the beam connection, a significant portion of the shear stress in the beam is actually transferred to the column through the beam flanges. This causes large localized secondary stresses in the beam flanges, both at the toe of the weld access hole and also in the complete joint penetration weld at the face of the column. The presence of the column web behind the column flange tends to locally stiffen the joint of the beam flange to the column flange, further concentrating the distribution of connection stresses and strains. Finally, the presence of the heavy beam and column flange plates, arranged in a "+" shaped pattern at the beam flange to column flange joint produces a condition of very high restraint, which retards the onset of yielding by raising the effective yield strength of the material, and allowing the development of very large stresses.

The most severe stresses typically occur at the root of the complete joint penetration weld of the beam bottom flange to the column flange. This is precisely the region of this welded joint that is most difficult for the welder to properly complete, as the access to the weld is restricted by the presence of the beam web and the welder often performs this weld while seated on the top flange, in the so-called "wildcat" position. The welder must therefore work from both sides of the beam web, starting and terminating the weld near the center of the joint, a practice that often results in poor fusion and the presence of slag inclusions at this location. These conditions, which are very difficult to detect when the weld backing is left in place, as was the typical practice, are ready-made crack initiators. When this region of the welded joints is subjected to the large concentrated tensile stresses, the weld defects begin to grow into cracks and these cracks can quickly become unstable and propagate as brittle fractures. Once these brittle fractures initiate, they can grow in a variety of patterns, as described above, under the influence of the stress field and the properties of the base and weld metals present at the zone of the fracture.

Despite the obvious local strength impairment resulting from these fractures, many damaged buildings did not display overt signs of structural damage, such as permanent drifts or extreme damage to architectural elements. Until news of the discovery of connection fractures in some buildings began to spread through the engineering community, it was relatively common for engineers to perform cursory post-earthquake evaluations of WSMF buildings and declare that they were undamaged. In order to reliably determine if a building has sustained connection damage, it is necessary to remove architectural finishes and fireproofing and perform ~~nondestructive examination including visual inspection and ultrasonic testing~~ careful visual inspection of the welded joints supplemented, in some cases, by nondestructive testing. Even if no damage is found, this is a costly process. Repair of damaged connections is even more costly. ~~A few WSMF buildings have sustained so much connection damage that it has been deemed more practical to demolish the structures rather than to repair them.~~ In the case of one WSMF building, damaged by the Northridge earthquake, repair costs were sufficiently large that the owner elected to demolish rather than replace than building.

Immediately following the Northridge Earthquake, a series of tests of beam-column subassemblies were performed at the University of Texas at Austin, under funding provided by the AISC as well as private sources. The test specimens used heavy W14 column sections and deep (W36) beam sections commonly employed in some California construction. Initial specimens were fabricated using the standard prequalified connection specified by the *Uniform Building Code* (*UBC*). Section 2211.7.1.2 of *UBC-94* {*NEHRP-91* Section 10 10.2.3} specified this prequalified connection as follows:

> "**2211.7.1.2 Connection strength**. The girder top column connection may be considered to be adequate to develop the flexural strength of the girder if it conforms to the following:
>
> 1 the flanges have full penetration butt welds to the columns.
>
> 2 the girder web to column connection shall be capable of resisting the girder shear determined for the combination of gravity loads and the seismic shear forces which result from compliance with Section 2211.7.2.1. This connection strength need not exceed that required to develop gravity loads plus $3(R_w/8)$ times the girder shear resulting from the prescribed seismic forces.

> Where the flexural strength of the girder flanges is greater than 70 percent of the flexural strength of the entire section, (i.e. $bt_f(d-t_f)F_y>0.7Z_xF_y$) the web connection may be made by means of welding or high-strength bolting.
>
> For girders not meeting the criteria in the paragraph above, the girder web-to-column connection shall be made by means of welding the web directly or through shear tabs to the column. That welding shall have a strength capable of developing at least 20 percent of the flexural strength of the girder web The girder shear shall be resisted by means of additional welds or friction-type slip-critical high strength bolts or both.
>
> and:
>
> **2211.7.2.1 Strength.** The panel zone of the joint shall be capable of resisting the shear induced by beam bending moments due to gravity loads plus 1.85 times the prescribed seismic forces, but the shear strength need not exceed that required to develop $0.8\Sigma M_s$ of the girders framing into the column flanges at the joint."

In order to investigate the effects that backing bars and weld tabs had on connection performance, these were removed from the specimens prior to testing. Despite these precautions, the test specimens failed at very low levels of plastic loading. Following these tests at the University of Texas at Austin, reviews of literature on historic tests of these connection types indicated a significant failure rate in past tests as well, although these had often been ascribed to poor quality in the specimen fabrication. It was concluded that the prequalified connection, specified by the building code, was fundamentally flawed and should not be used for new construction in the future.

In retrospect, this conclusion may have beensomewhat premature. More recent testing of connections having configurations similar to those of theprequalified connection, but incorporating tougher weld metals, having backing bars removed from the bottom flange jointand fabricated with greater care to avoid the defects that can result in crack initiation, have performed better than those initially tested at the University of Texas. However, as a class, when fabricated using currently prevailing construction practice, these connectionsstill do not appear to be capable of consistently developing the levels of ductility presumed by the building codes for service in moment-resisting frames that are subjected to large inelastic demands~~When the first test specimens for that series were fabricated, the welder failed to follow the intended welding procedures. Further, no special precautions were taken to assure that the materials incorporated in the work had specified toughness. Some engineers, with knowledge of fracture mechanics, have suggested that if materials with adequate toughness are used, and welding procedures are carefully specified and followed, adequate reliability can be obtained from the traditional connection details. Others believe that the conditions of high tri-axial restraint present in the beam flange to column flange joint (Blodgett - 1995) would prevent ductile behavior of these joints regardless of the procedure used to make the welds. Further they point to the important influence of the relative yield and tensile strengths of beam and column materials, and other variables, that can affect connection behavior. To date, there has not been sufficient research conducted to resolve this issue.~~

In reaction to the University of Texas tests as well as the widespread damage discovered following the Northridge Earthquake, and the urging of the California Seismic Safety Commission, in September, 1994 the International Conference of Building Officials (ICBO) adopted an emergency code change to the 1994 edition of the *Uniform Building Code* (*UBC*-94) {1994 *NEHRP Recommended Provisions Section 5.2*} This code change, jointly developed by the Structural Engineers Association of California, AISI and ICBO staff, deleted theprequalified connection and substituted the following in its place.

> "**2211.7.1.2 Connection Strength**. Connection configurations utilizing welds or high-strength bolts shall demonstrate, by approved cyclic test results or calculation, the ability to sustain inelastic rotation and develop the strength criteria in Section 2211.7.1.1 considering the effect of steel overstrength and strain hardening."
>
> "**2211.7.1.1 Required strength**. The girder-to-column connection shall be adequate to develop the lesser of the following.
>
> 1 The strength of the girder in flexure.
>
> 2 The moment corresponding to development of the panel zone shear strength as determined from formula 11-1."

Unfortunately, neither the required "inelastic rotation", or calculation and test procedures are well defined by these code provisions. Design Advisory No. 3 (SAC-1995) included an Interim Recommendation (SEAOC-1995) that attempted to clarify the intent of this code change, and the preferred methods of design in the interim period until additional research could be performed and reliable acceptance criteria for designs re-established. The State of California similarly published a joint Interpretation of Regulations (DSA-OSHPD - 1994) indicating the interpretation of the current code requirements which would be enforced by the state for construction under its control. This applied only to the construction of schools and hospitals in the State of California. The intent of these Interim Guidelines is to supplement these previously published documents and to provide updated recommendations based on the results of the limited directed research performed to date.

1.4 The SAC Joint Venture

There are no modifications to the Guidelines or Commentary of Section 1.4 at this time.

1.5 Sponsors

There are no modifications to the Guidelines or Commentary of Section 1.5 at this time.

1.6 Summary of Phase 1 Research

There are no modifications to the Guidelines or Commentary of Section 1.6 at this time.

1.7 Intent

There are no modifications to the Guidelines or Commentary of Section 1.7 at this time.

1.8 Limitations

There are no modifications to the Guidelines or Commentary of Section 1.8 at this time.

1.9 Use of the Guidelines

There are no modifications to the Guidelines or Commentary of Section 1.9 at this time

3. CLASSIFICATION AND IMPLICATIONS OF DAMAGE

3.1 Summary of Earthquake Damage

There are no modifications to the Guidelines or Commentary of Section 3.1 at this time.

3.2 Damage Types

There are no modifications to the Guidelines or Commentary of Section 3.2 at this time

3.2.1 Girder Damage

There are no modifications to the Guidelines or Commentary of Section 3.2.1 at this time.

3.2.2 Column Flange Damage

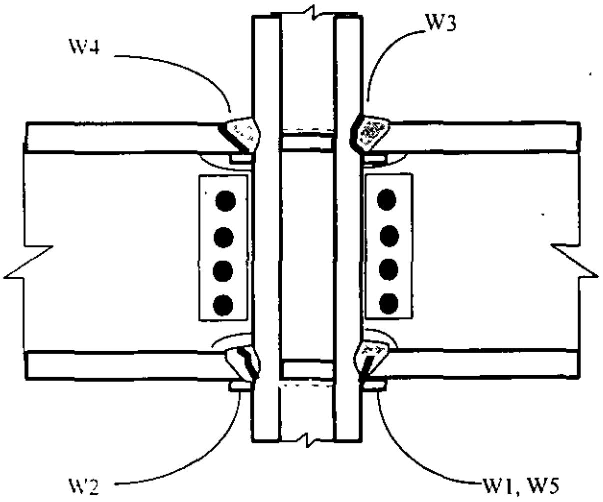
There are no modifications to the Guidelines or Commentary of Section 3.2.2 at this time.

3.2.3 Weld Damage, Defects and Discontinuities

Six types of weld discontinuities. defects and damage are defined in Table 3-3 and illustrated in Figure 3-4. All apply to the complete joint penetration (CJP) welds between the girder flanges and the column flanges. ~~This category of damage was the most commonly reported type (f~~Following the Northridge Earthquake, many instances of W1a and W1b conditions were reported as damage. These conditions, which are detectable only by ultrasonic testing or by removal of weld backing, are now thought more likely to be construction defects than damage.

Table 3-3 - Types of Weld Damage, Defects and Discontinuities

Type	Description
W1	Weld root indications
W1a	Incipient indications-– depth <3/16" or $t_f/4$; width < $b_f/4$
W1b	Root indications larger than that for W1a
W2	Crack through weld metal thickness
W3	Fracture at column interface
W4	Fracture at girder flange interface
W5	UT detectable indication - nonrejectable



Note. See Figure 3-2 for related column damage and Figure 3-3 for girder damage

Figure 3-4 - Types of Weld Damage

Commentary: Despite significant controversy, type W1 and W5 discovered in buildings following the Northridge earthquake, were commonly reported as damage. These small discontinuities and defects located at the roots of the CJP welds are detectable only by ultrasonic testing (UT) when the weld backing is left in place or by visual testing (VT) or magnetic particle testing (MT) when weld backing is removed. It now seems likely that most such conditions are not damage at all, but rather, are pre-existing construction defects. A number of factors point to this conclusion. First, statistical surveys of damage sustained by buildings in the Northridge earthquake show that if type W1 and W5 conditions are not considered, there was a much greater incidence of damage in frames resisting north-south ground shaking than in frames resisting east-west shaking. This appears to be correlated with the relative strength of the ground shaking experienced along these two directional axes. However, there is no significant difference between the incidence rate of reported W1 and W5 conditions in these two directions, suggesting that these conditions are not correlated with shaking intensity.

The discovery of W1 conditions in welds for which original construction quality assurance documentation is available, indicating that no such defects were present when the building was originally constructed, tends to contradict this argument. However, investigations conducted by SAC under the Phase 2 project have indicated that as a result of the joint geometry, UT techniques are often unable to detect W1 conditions at the weld root, when scanning of the joint is conducted from the top surface of the beam bottom flange. It is important to note that this is the most common method of conducting UT as part of construction quality assurance. When UT scanning of a joint is conducted from the bottom surface of the flange, as is commonly done when inspecting for *earthquake damage, it becomes more likely that such conditions will be detected, since the geometric constraints present for top flange scanning are altered. This leads to the conclusion that it is probable that typical construction quality assurance UT of welded joints would be likely to miss W1 conditions, allowing them to be discovered in later post-earthquake surveys.*

When FEMA-267 was first published, it was recommended that W1 conditions be treated as damage and that UT be used as a routine part of the post-earthquake investigation process, in order to discover these conditions. However, more recent investigations conducted by SAC have revealed that even the careful scanning typically conducted as part of a post-earthquake inspection is not able to reliably detect these conditions. Given that it is both expensive and difficult to locate W1 conditions as part of a post-earthquake investigation, and also, that most of these conditions are unlikely to be damage at all, it is no longer recommended that exhaustive investigations for these conditions be conducted as part of the earthquake damage investigation process.

~~*Type W1 damage discontinuities and defects and type W5 discontinuities are detectable only by NDT, unless the backing bar is removed, allowing direct detection by visual inspection or magnetic particle testing. Type W5 consists of small discontinuities and may or may not actually be earthquake damage. AWS D1.1 permits small discontinuities in welds. Larger discontinuities are termed defects, and are rejectable per criteria given in the Welding Code. It is likely [illegible] that some weld indications detected by NDT in a post-earthquake inspection may be discontinuities which pre-existed the earthquake and do not constitute a rejectable condition, per the AWS standards. Repair of these discontinuities designated as type W5 is not generally recommended. Some type W1 indications are small planar defects, which are rejectable per the AWS D1.1 criteria but are not large enough to be classified as one of the types W2 through W4. Type W1 is the single most commonly reported non-conforming condition reported in the post-Northridge statistical data survey, and in some structures, accounted for more than 80 per cent of the total damage reported. The W1 classification is split into two types, W1a and W1b, based on their severity. Type W1a [illegible] root indications are defined as being nominal in extent, less than [illegible] deep or [illegible] the flange thickness, whichever is less, and having a length [illegible] of the flange width. Some engineers believe that type W1a indications are not earthquake damage at all, but rather, previously undetected defects from the original construction process. A W1b indication is one that exceeds the W1a limits but is not clearly characterized by one of the other types. It is more likely that W1b indications are a result of the earthquake than the construction process.*~~

~~*As previously stated, some engineers believe that both type W1a and some type W1b conditions are not earthquake related damage at all, but instead, are*~~

~~*rejectable conditions not detected by the quality control and assurance programs in effect during the original construction. However, in recent large scale sub-assembly testing of the inelastic rotation capacity of girder-column connections conducted in SAC Phase 1 at the University of Texas at Austin and the Earthquake Engineering Research Center of the University of California at Berkeley, it was reported that signficantly more indications were detectable in unfailed CJP welds following the testing than were detectable prior to the test. This tends to indicate that type W1 damage may be related to stresses induced in the structures by their response to the earthquake ground motions. Regardless of whether or not type W1 conditions are directly attributable to earthquake response, it is clear that these conditions result in a reduced capacity for the CJP welds and can act as stress risers, or notches, to initiate fracture in the event of future strong demands.*~~

Type W2 fractures extend completely through the thickness of the weld metal and can be detected by either MT or VI techniques. Type W3 and W4 fractures occur at the zone of fusion between the weld filler metal and base material of the girder and column flanges, respectively. All three types of damage result in a loss of tensile capacity of the girder flange to column flange joint and should be repaired.

As with girder damage, damage to welds has most commonly been reported at the bottom girder to column connection, with fewer instances of reported damage at the top flange. Available data indicates that approximately 25 per cent of the total damage in this category occurs at the top flange, and most often, top flange damage occurs in connections which also have bottom flange damage. For the same reasons previously described for girder damage, less weld damage may be expected at the top flange. However, it is likely that there is a significant amount of damage to welds at the top girder flange which have never been discovered due to the difficulty of accessing this joint. Later sections of these Interim Guidelines provide recommendations for situations when such inspection should be performed.

3.2.4 Shear Tab Damage

There are no modifications to the Guidelines or Commentary of Section 3.2.4 at this time.

3.2.5 Panel Zone Damage

There are no modifications to the Guidelines or Commentary of Section 3.2.5 at this time.

3.2.6 Other Damage

There are no modifications to the Guidelines or Commentary of Section 3.2.6 at this time.

3.3 Safety Implications

The implications of the damage described above with regard to building safety are discussed in this section. As part of the SAC Phase 2 program, extensive nonlinear analyses have been conducted of WSMF buildings to determine the effects of connection fractures on building performance and also to develop an understanding of the risk of earthquake-induced building collapse. These studies indicate that risk of collapse of WSMF buildings designed to modern standards and having connections capable of ductile behavior is quite low. Even in regions of very high seismicity, such as those areas of coastal California adjacent to major active faults, the probability that such a building would experience earthquake-induced collapse appears to be on the order of one occurrenceper building, every20,000 years. For buildings that have brittle connections such as those commonly constructed prior to 1994, the probability of collapse increases somewhat. If only the bottom flange connections of beams to columns is subject to fracture, the risk of global collapse of buildings increases to perhaps one occurrence in 15,000 years, presuming that the fractures do not jeopardize column capacity. However, if both flanges of the connections are subject to fracture, or if substantial column damage occurs, the risk of collapse increases significantly. Also, it is important to note that severe connection fractures can result in significant risk of local collapse and life safety endangerment.

While these studies have been helpful in providing an understanding of the level of risk inherent in WSMF structures with brittle connections they do not provide sufficient information to ~~There is insufficient knowledge at this time to permit determination of the~~assess the degree of risk with any real confidence However, based on the historic performance of modern WSMF buildings, typical of those constructed in the United States, it appears that the risk of collapse in moderate magnitude earthquakes, ranging up to perhaps M7, isvery low for buildings which have been properly designed and constructed according to prevailing standards. A possible exception to this may be buildings located in the near field (< 10 km from the surface projection of the fault rupture) of such earthquakes (Heaton, et. al - 1995), however, this is not uniquely a problem associated with steel buildings. Our current building codes ingeneral, may not be adequate to provide for reliable performance of buildings within the near field of large earthquakes. As is also the case with all other types of construction, buildings with incomplete lateral force resisting systems, severe configuration irregularities, inadequate strength or stiffness, poor construction quality, or deteriorated condition are at higher risk than buildings not possessing these characteristics.

No modern WSMF buildings have been sited within the areas of very strong ground motion from earthquakes larger than M7, or for that matter, within the very near field for events exceeding M6.5. This style of construction has been in wide use only in the past few decades. Consequently, it is not possible to state what level of risk may exist with regard to building response to such events. This same lack of performance data for large magnitude, long duration events exists for virtually all forms of contemporary construction. Consequently, there is considerable uncertainty in assigning levels of risk to any building designed to minimum code requirements for these larger events.

Commentary: ~~Research conducted to date has not been conclusive with regard to the risk of collapse of WSMF buildings. Some testing of damaged connections from a building in Santa Clarita, California have been conducted at the University of Southern California (Anderson - 1995). In these tests, connection assemblies which had experienced type P6 damage were subjected to repeated cycles of flexural loading, while the column was maintained under axial compression. Under these conditions, the specimens were capable of resisting as much as 10 per cent of the nominal plastic strength of the girder for several cycles of slowly applied loading, at plastic deformation levels as large as 0.025 radians. However, damage did progress in the specimen, as this testing was performed. It is not known how these assemblies would have performed if the columns were permitted to experience tensile loading. Data from other tests suggests that the residual strength of connections which have experienced types G3, G4, W2, W3, and W4 damage is on the order of 15 per cent of the undamaged strength. Some analytical research (Hall - 1995) in which nonlinear time history analyses simulating the effects of connection degradation due to fractures were included, indicates that typical ground motions resulting in the near field of large earthquakes can cause sufficient drift in these structures to induce instability and collapse. Other researchers (Astaneh - 1995) suggest that damaged structures, even if unrepaired, have the ability to survive additional ground motion similar to that of the Northridge Earthquake.~~

Even though there were no collapses of WSMF buildings in the 1994 Northridge Earthquake, it should not be assumed that no risk of such collapse exists. Indeed, a number of WSMF buildings did experience collapse in the 1995 Kobe Earthquake. The detailing of these collapsed Japanese buildings was somewhat different than that found in typical US practice, however, much of the fracture damage that occurred was similar to that discovered following the Northridge event.

Because of a lack of data and experience with the effects of larger, longer duration earthquakes, there is considerable uncertainty about the performance of all types of buildings in large magnitude seismic events. It is believed that seismic risks in such large events are highly dependent on the individual ground motion at a specific site and the characteristics of the individual buildings. Therefore, generalizations with regard to the probable performance of individual types of construction may not be particularly meaningful.

The risks to occupants of WSMF buildings with brittle connections is regarded as less, in most cases, than to occupants of the types of buildings listed below. However, because of the uncertainties involved, the degree of risk in large events cannot be definitively quantified, nor can it categorically be stated that properly constructed WSMF buildings sited in the near field of large events are either *more or less at risk than many other code designed building systems which do not appear on the following list:*

- *Concentric braced steel frames with bracing connections that are weaker than the braces*
- *Knee braced steel frames*
- *Unreinforced masonry bearing wall buildings*
- *Non-ductile reinforced concrete moment frames (infilled or otherwise)*
- *Reinforced concrete moment frames with gravity load bearing elements that were not designed to participate in the lateral force resisting system and that do not have capacity to withstand earthquake-induced deformations*
- *Tilt-up and reinforced masonry buildings with inadequate anchorage of their heavy walls to their horizontal wood diaphragms*
- *Precast concrete structures without adequate interconnection of their structural elements*

In addition, WSMF structures with brittle connections would appear to have lower inherent seismic risk than structures of any construction type that:

- *do not having complete, definable load paths*
- *have significant weak and/or soft stories*
- *have major torsional irregularity and insufficient stiffness and strength to resist the resulting seismic demands*
- *minimal redundancy and concentrations of lateral stiffness*

These are general statements that represent a global view of system performance. As with all seismic performance generalizations, there are many steel moment frame buildings that are more vulnerable to damage than some individual buildings of the general categories listed, just as there are many that will perform better.

3.4 Economic Implications

There are no modifications to the Guidelines or Commentary of Section 3.4 at this time.

4. POST-EARTHQUAKE EVALUATION

4.1 Scope

There are no modifications to the Guidelines or Commentary of Section 4.1 at this time.

4.2 Preliminary Evaluation

There are no modifications to the Guidelines or Commentary of Section 4.2 at this time.

4.2.1 Evaluation Process

Preliminary evaluation is the process of determining if a building should be subjected to detailed post-earthquake evaluations. Detailed evaluations should be performed for all buildings thought to have experienced strong ground motion, as indicated in Section 4.2.1.1 or for which the other indicators of Section 4.2.1.2 apply. Detailed post-earthquake evaluations include the entire process of determining if a building has experienced significant damage and if damage is found, determining appropriate strategies for occupancy, structural repair and/or modification. Except as indicated in Section 4.2 3, detailed evaluation should as a minimum, include inspections of a representative sample ofmoment-resisting (and other type) connections within the building.

4.2.1.1 Ground Motion

There are no modifications to the Guidelines or Commentary of Section 4.2.1.1 at this time.

4 2.1.2 Additional Indicators

There are no modifications to the Guidelines or Commentary of Section 4.2.1.2 at this time.

4.2.2 Evaluation Schedule

There are no modifications to the Guidelines of Section 4.2.2 at this time.

> *Commentary: It is important to conduct post-earthquake evaluations as soon following the earthquake as is practical. Aftershock activity in the months immediately following an earthquake is likely to produce additional strong ground motion at the site of a damaged building. If there is adequate reason to assume that damage has occurred, then such damage should be expeditiously uncovered and repaired. However, since adequate resources for post-earthquake evaluation may be limited, a staggered schedule is presented, with those buildings having a greater likelihood of damage recommended for evaluation first.*

Large magnitude earthquakes are often followed by large magnitude aftershocks. Therefore, it is particularly urgent that post-earthquake evaluations be performed expeditiously following such events. ~~*If insufficient resources are available in the affected region to perform the NDT tests recommended by the Guidelines of Chapter 5, it is recommended that visual inspection, in accordance with Section 5.2.2, proceed as soon as possible*~~*. If visual inspection reveals substantial damage, consideration should be given to vacating the building until either an adequate period of time has passed so as to make the likelihood of very large aftershocks relatively low (e.g. 4 weeks for magnitude 7 and lower, and 8 weeks for magnitudes above this), complete inspections and repairs are made, or a detailed evaluation indicates that the structure retains adequate structural stiffness and strength to resist additional strong ground shaking.* ~~*Preliminary visual inspections should not be used as an alternative to complete evaluation.*~~

~~*The table*~~ *Table 4-1 relates the urgency for post-earthquake building evaluation to both the magnitude of the earthquake and the estimated peak ground acceleration experienced by the building site. This is because large magnitude events are more likely to have large magnitude aftershocks and because buildings that experienced stronger ground accelerations are more likely to have been damaged. Except in regions with extensive strong motion instrumentation, estimates of ground motion are quite subjective. Following major damaging earthquakes, government agencies usually produce ground motion maps showing projected acceleration contours. These maps should be used when available. When such maps are not available, ground motions can be estimated using any of several attenuation relationships that have been published.*

4.2.3 Connection Inspections

Except as indicated in Sections 4.2.3.1 and 4.2.3.2, below, ~~D~~detailed evaluations should include inspection of the building's moment-resisting connections in order to determine their condition. As a first pass, inspections may be limited to careful visual inspection of the joint of the beam bottom flange to the column. When such inspection reveals the presence of connection damage, a more thorough inspection of the damaged connection should be conducted. Since moment-resisting frame buildings commonly have many connections, inspections can be quite costly. Therefore, it shall be permissible to limit inspections to~~of~~ a representative sample of WSMF (and other) connections~~, except as indicated in Sections 4.2.3.1 and 4.2.3.2, below.~~ Section 4.3.3 provides three alternative approaches to selecting an appropriate sample of connections for inspection.

4.2.3.1 Analytical Evaluation

There are no modifications to the Guidelines or Commentary of Section 4.2.3.1 at this time.

4.2.3.2 Buildings with Enhanced Connections

There are no modifications to the Guidelines or Commentary of Section 4.2.3.2 at this time.

4.2.4 Previous Evaluations and Inspections

There are no modifications to the Guidelines or Commentary of Section 4.2.4 at this time.

4.3 Detailed Evaluation Procedure

Where detailed evaluation is recommended by Section 4.2, assessment of the post earthquake condition of a building, its ability to resist additional strong ground motion and other loads, and determination of appropriate occupancy, structural repair and/or modification strategies should be based on the results of a detailed inspection and assessment of the extent to which structural systems have been damaged.

In order to obtain complete data on a building's post-earthquake condition, it is necessary to inspect each of the building's moment-resisting frame elements and their connections. However, such extensive inspections could be very costly. As an alternative to that approach, this Section presents a series of procedures by which a representative sample of beam-column connections is selected and inspected. ~~This Section presents one approach for making such assessments~~. In this approach, the results of the sample inspections are used to calculate a cumulative damage index, D, for the structure as well as the probability that if all of the building's connections had been inspected, the damage index at any floor of the structure ~~has~~ would have been found to exceed~~ed~~ a value of 1/3. General occupancy, structural repair and modification recommendations are made based upon the values calculated for these damage indices. In particular, a calculated damage index of 1/3 is used to indicate, in the absence of more detailedanalyses, that a potentially hazardous condition may exist.

The structural engineer may use other procedures consistent with the principles of statistics and structural mechanics to determine the residual strength and stiffness of the structure in the as-damaged state and the acceptability of such characteristics relative to the criteria contained in the building code, or other rational criteria acceptable to the building official.

There are no modifications to the Commentary of Section 4.3 at this time.

4.3.1 Eight Step Evaluation Procedure

Post-earthquake evaluation should be carried out under the direct supervision of a structural engineer. The following eight-step procedure may be used to determine the condition of the structure and to develop occupancy, repair and modification strategies. Note that this procedure is written presuming that inspection is limited to a representative sample of the total number of connections present in the building. If all connections in the building are to be inspected, steps 1, 2, 4 and 6 may be omitted.

Step 1: The moment-resisting connections in the building are categorized into two or more "groups" (Section 4.3.2 and 4.4) comprised of connections expected to have similar probabilities of being damaged.

Complete steps 2 through 7 below, for each group of connections.

Step 2: Determine the minimum number of connections in each group that should be inspected and select the specific sample of connections to be inspected. (Section 4.3.3)

Step 3 Inspect the selected set of connections using the technical guidelines of Section 5.2. and determine connection damage indices,d_j, for each inspected connection (Section 4.3.4)

Step 4: If inspected connections are found to be seriously damaged, perform additional inspections of connections adjacent to the damaged connections. (Section 4.3.5)

Step 5: Determine the average damage index (d_{avg}) for connections in each group, and then the average damage index at a typical floor. (Section 4.3.6)

Step 6 Given the average damage index for connections in the group, determine the probability, P, that the connection damage index for any group, at a floor level, exceeds 1/3, and determine the maximum estimated damage index for any floor, D_{max}. (Section 4.3.7)

Step 7: Based on the calculated damage indices and statistics, determine appropriate occupancy, structural repair and modification strategies (Section 4.3.8). If deemed appropriate, the structural engineer may conduct detailed structural analyses of the building in the as-damaged state, to obtain improved understanding of its residual condition and to confirm that the recommended strategies are appropriate or to suggest alternative strategies.

Step 8. Report the results of the inspection and evaluation process to the building official and building owner. (Section 4.3.9)

Sections 4.3.2 through 4.3.9 indicate how these steps should be performed.

There are no modifications to the Commentary of Section 4.3.1 at this time.

4.3.2 Step 1— Categorize Connections by Groups

There are no modifications to the Guidelines or Commentary of Section 4.3.2 at this time.

4.3.3 Step 2— Select Samples of Connections for Inspection

There are no modifications to the Guidelines or Commentary of Section 4.3.3 at this time.

4.3.3.1 Method A - Random Selection

There are no modifications to the Guidelines or Commentary of Section 4.3.3.1 at this time.

4.3.3.2 Method B - Deterministic Selection

There are no modifications to the Guidelines or Commentary of Section 4.3.3.2 at this time.

4.3.3.3 Method C - Analytical Selection

There are no modifications to the Guidelines or Commentary of Section 4.3.3.3 at this time.

4.3.4 Step 3— Inspect the Selected Samples of Connections

There are no modifications to the Guidelines of Section 4.3.4 at this time.

> ~~*Commentary: The sample size suggested for inspection in the methods of Section 4.3.3 are based on full inspection using both visual (Section 5.3.1) and NDT techniques (Section 5.3.2) at all connections in the sample. Other methods of selection and inspection may be used as provided in Section 4.3, with the approval of the building official. One such approach might be the visual only inspection of the bottom girder flange to column connection, but with the inspection of a large fraction of the total connections in the group, possibly including all of them. If properly performed, such an inspection procedure would detect almost all instances of the most severe damage but would not detect weld defects (W1a), or root cracking (W1b), nor lamellar damage in columns (C5). The occurrence of a few of these conditions, randomly scattered through the building would not greatly affect the assessment of the building's post-earthquake condition, or the calculation of the damage index. However, if a large number of such defects were present in the building, this would be significant to the overall assessment. Therefore, such an inspection approach should probably include confirming NDT investigations of at least a representative sample of the total connections investigated. If within that sample, significant incidence of visually hidden damage is found, then full NDT investigations should be performed, as suggested by these Interim Guidelines. Similarly, if visual damage is found at the bottom flange, then complete connection inspection should be performed to determine if other types of damage are also present.*~~

4.3.4.1 Damage Characterization

Characterize the observed damage at each of the inspected connections by assigning a connection damage index, d_j, obtained either from Table 4-3a or Table 4-3b. Table 4-3a presents damage indices for individual classes of damage and a rule for combining indices where a connection has more than one type of damage. Table 4-3b provides combined indices for the more common combinations of damage.

Table 4-3a - Connection Damage Indices

Type	Location	Description[1]	Index[2] d_j
G1	Girder	Buckled Flange	4
G2	Girder	Yielded Flange	1
G3	Girder	Top or Bottom Flange fracture in HAZ	8
G4	Girder	Top or Bottom Flange fracture outside HAZ	8
G5	Girder	Top and Bottom Flange fracture	10
G6	Girder	Yielding or Buckling of Web	4
G7	Girder	Fracture of Web	10
G8	Girder	Lateral-torsional Buckling	8
C1	Column	Incipient flange crack (detectable by UT)	4
C2	Column	Flange tear-out or divot	8
C3	Column	Full or partial flange crack outside HAZ	8
C4	Column	Full or partial flange crack in HAZ	8
C5	Column	Lamellar flange tearing	6
C6	Column	Buckled Flange	8
C7	Column	Fractured column splice	8
W1a	CJP weld	Minor root indication - thickness <3/16" or $t_f/4$; width < $b_f/4$	0~~4~~
W1b	CJP weld	Root indication - thickness > 3/16" or $t_f/4$ or width > $b_f/4$	0~~4~~
W2	CJP weld	Crack through weld metal thickness	8
W3	CJP weld	Fracture at girder interface	8
W4	CJP weld	Fracture at column interface	8
W5	CJP weld	Root indication— non-rejectable	0
S1a	Shear tab	Partial crack at weld to column (beam flanges sound)	4
S1b	Shear tab	Partial crack at weld to column (beam flange cracked)	8
S2a	Shear tab	Crack in Supplemental Weld (beam flanges sound)	1
S2b	Shear tab	Crack in Supplemental Weld (beam flange cracked)	8
S3	Shear tab	Fracture through tab at bolt holes	10
S4	Shear tab	Yielding or buckling of tab	6
S5	Shear tab	Damaged, or missing bolts[4]	6
S6	Shear tab	Full length fracture of weld to column	10
P1	Panel Zone	Fracture, buckle, or yield of continuity plate[3]	4
P2	Panel Zone	Fracture of continuity plate welds[3]	4
P3	Panel Zone	Yielding or ductile deformation of web[3]	1
P4	Panel Zone	Fracture of doubler plate welds[3]	4
P5	Panel Zone	Partial depth fracture in doubler plate[3]	4
P6	Panel Zone	Partial depth fracture in web[3]	8
P7	Panel Zone	Full (or near full) depth fracture in web or doubler plate[3]	8
P8	Panel Zone	Web buckling[3]	6
P9	Panel Zone	Fully severed column	10

Notes To Table 4-3a

1. See Figures 3-2 through 3-6 for illustrations of these types of damage.
2. Where multiple damage types have occurred in a single connection, then:
   a. Sum the damage indices for all types of damage with d=1and treat as one type If multiple types still exist, then

b. For two types of damage refer to Table 4-3b. If the combination is not present in Table 4-3b and the damage indices for both types are greater than or equal to 4, use 10 as the damage index for the connection. If one is less than 4, use the greater value as the damage index for the connection

c. If three or more types of damage apply and at least one is greater than 4, use an index value of 10, otherwise use the greatest of the applicable individual indices.

3 Panel zone damage should be reflected in the damage index for all moment connections attached to the damaged panel zone within the assembly.

4 Missing or loose bolts may be a result of construction error rather than damage. The condition of the metal around the bolt holes, and the presence of fireproofing or other material in the holes can provide clues to this. Where it is determined that construction error is the cause, the condition should be corrected and a damage index of "0" assigned

Table 4-3b - Connection Damage Indices for Common Damage Combinations[1]

Girder, Column or Weld Damage	Shear Tab Damage	Damage Index
G3 or G4	S1a	8
	S1b	10
	S2a	8
	S2b	10
	S3	10
	S4	10
	S5	10
	S6	10
C2	S1a	8
	S1b	10
	S2a	8
	S2b	10
	S3	10
	S4	10
	S5	10
	S6	10
C3 or C4	S1a	8
	S1b	10
	S2a	8
	S2b	10
	S3	10
	S4	10
	S5	10
	S6	10
C5	S1a	6
	S1b	10
	S2a	6
	S2b	10
	S3	10
	S4	10
	S5	10
	S6	10
W2, W3, or W4	S1a	8
	S1b	10
	S2a	8
	S2b	10
	S3	10
	S4	10
	S5	10
	S6	10

1 See Table 4-3a, footnote 2 for combinations other than those contained in this table.

More complete descriptions (including sketches) of the various types of damage are provided in Section 3.1. When the engineer can show by rational analysis that other values for the relative severities of damage are appropriate, these may be substituted for the damage indices provided in the tables. A full reporting of the basis for these different values should be provided to the building official, upon request.

> *Commentary: The connection damage indices provided in Table 4-3 (ranging from 0 to 10) represent judgmental estimates of the relative severities of this damage. An index of 0 indicates no damage and an index of 10 indicates very severe damage.*
>
> *When initially developed, these connection damage indices were conceptualized as estimates of the connection's lost capacity to reliably participate in the building's lateral-force-resisting system in future earthquakes (with 0 indicating no loss of capacity and 10 indicating complete loss of capacity). However, due to the limited data available, no direct correlation between these damage indices and the actual residual strength and stiffness of a damaged connection was ever made. They do provide a convenient measure, however, of the extent of damage that various connections in a building have experienced.*
>
> *When FEMA-267 was first published, weld root discontinuities, Type W1a and defects, type W1b, were classified as damage in Table 4-3a with damage indices of 1 and 4, respectively assigned. Recent evidence and investigations, however, suggest strongly that these W1 conditions are not likely to be damage, and also are difficult to reliably detect. As a result, with the publication of Interim Guidelines Advisory No. 2, the damage indices for these conditions has been reduced to a null value, consistent with classifying them as pre-existing conditions, rather than damage.*
>
> *It should be noted that the reduced damage index associated with these conditions is not intended to indicate that these are not a concern with regard to future performance of the building. In particular, type W1b conditions can serve as ready initiators for the types of brittle fractures associated with the other damage types and connections having such conditions are more susceptible to future earthquake-induced damage than connections that do not have these conditions. Correction of these conditions should generally be considered an upgrade or modification, rather than a damage repair.*

4.3.5 Step 4— Inspect Connections Adjacent to Damaged Connections

There are no modifications to the Guidelines or Commentary of Section 4.3.5 at this time.

4.3.6 Step 5— Determine Average Damage Index for Each Group

There are no modifications to the Guidelines or Commentary of Section 4.3.6 at this time.

4.3.7 Step 6— Determine the Probability that the Connections in a Group at a Floor Level Sustained Excessive Damage

There are no modifications to the Guidelines or Commentary of Section 4.3.7 at this time.

4.3.7.1 Some Connections in Group Not Inspected

There are no modifications to the Guidelines or Commentary of Section 4.3.7.1 at this time.

4.3.7.2 All Connections in Group Inspected

There are no modifications to the Guidelines or Commentary of Section 4.3.7.2 at this time.

4.3.8 Step 7— Determine Recommended Recovery Strategies for the Building

There are no modifications to the Guidelines or Commentary of Section 4.3.8 at this time.

4.3.9 Step 8 - Evaluation Report

There are no modifications to the Guidelines or Commentary of Section 4.3.9 at this time.

4.4 Alternative Group Selection for Torsional Response

There are no modifications to the Guidelines or Commentary of Section 4.4 at this time.

4.5 Qualified Independent Engineering Review

There are no modifications to the Guidelines or Commentary of Section 4.5 at this time.

4.5.1 Timing of Independent Review

There are no modifications to the Guidelines or Commentary of Section 4.5.1 at this time.

4.5.2 Qualifications and Terms of Employment

There are no modifications to the Guidelines or Commentary of Section 4.5.2 at this time.

4.5.3 Scope of Review

There are no modifications to the Guidelines or Commentary of Section 4.5.3 at this time.

4.5.4 Reports

There are no modifications to the Guidelines or Commentary of Section 4.5.4 at this time.

4.5.5 Responses and Corrective Actions

There are no modifications to the Guidelines or Commentary of Section 4.5.5 at this time.

4.5.6 Distribution of Reports

There are no modifications to the Guidelines or Commentary of Section 4.5.6 at this time.

4.5.7 Engineer of Record

There are no modifications to the Guidelines or Commentary of Section 4.5.7 at this time

4.5.8 Resolution of Differences

There are no modifications to the Guidelines or Commentary of Section 4.5.8 at this time.

5. POST-EARTHQUAKE INSPECTION

When required by the building official, or recommended by the Interim Guidelines in Chapter 4, post-earthquake inspections of buildings may be conducted in accordance with the Interim Guidelines of this Chapter. In order to determine, with certainty, the actual post-earthquake condition of a building, it is necessary to inspect all elements and their connections. However, it is permissible to select ~~An~~ an appropriate sample (or samples) of WSMF connections ~~should be selected~~ for inspection in accordance with the Chapter 4 Guidelines. These connections, and others deemed appropriate by the engineer, should be subjected to visual inspection (VI) ~~and~~ supplemented by non-destructive testing (NDT) as required by this Chapter.

> *Commentary: The only way to be certain that all damage sustained by a building is detected is to perform complete inspections of every structural element and connection. In most cases, such exhaustive post-earthquake inspections would be both economically impractical and also unnecessary. As recommended by these guidelines, the purpose of post-earthquake inspections is not to detect all damage that has been sustained by a building, but rather, to detect with reasonable certainty, that damage likely to result in a significant degradation in the building's ability to resist future loading. The connection sampling process, suggested by Chapter 4 of these Interim Guidelines was developed to provide a low probability that damage in buildings that had sustained a substantial reduction in load carrying capacity would be overlooked while avoiding the performance of exhaustive investigations of buildings that have sustained relatively insignificant damage.*
>
> *Where greater certainty in the detection of damage is desired for a building, a more extensive program of inspection can be conducted. For those cases in which it is desired to perform an analytical determination of the residual load carrying capacity of the structure, complete inspections of elements and connections should be performed so that an analytical model of the building can be developed that reasonably represents its post-earthquake condition.*

5.1 Connection Types Requiring Inspection

5.1.1 Welded Steel Moment Frame (WSMF) Connections

The inspection of a WSMF connection should start with visual inspection of the welded bottom beam flange to column flange joint and the base materials immediately adjacent to this joint. If damage to this joint is apparent, or suspected, then inspections of that connection should be extended to include the complete joint penetration (CJP) groove welds connecting both top and bottom beam flanges to the column flange, including the backing bar and the weld access holes in the beam web; the shear tab connection, including the bolts, supplemental welds and beam web; the column's web panel zone, including doubler plates; and the continuity plates and continuity plate welds (See Figure 3-1). In addition, where visual inspection indicates potential concealed damage, visual inspection should be supplemented with other methods of nondestructive testing.

> *Commentary: The largest concentration of reported damage following the Northridge Earthquake occurred at the welded joint between the bottom girder flange and column, or in the immediate vicinity of this joint. To a much lesser extent, damage was also observed in some buildings at the joint between the top girder flange and column. If damage at either of these locations is substantial (d_i per Chapter 4 greater than 5), then damage is also commonly found in the panel zone or shear tab areas.*
>
> *When originally published, ~~These~~ these Interim Guidelines recommended complete inspection, by visual and NDT assisted means, of all of these potential damage areas for a small representative sample of connections. This practice ~~is~~ was consistent with that followed by most engineers in the Los Angeles area, following the Northridge Earthquake. It requires removal of fireproofing from a relatively large surface of the steel framing, which at most connections will be undamaged.*
>
> *In the time since the Interim Guidelines were first published, extensive investigations have been conducted of the statistical distribution of damage sustained by buildings in the Northridge earthquake, the nature of this damage and the effect of this damage on the future load-carrying capacity of the buildings. These investigations strongly suggest that the W1a and W1b conditions at the weld root are unlikely to be earthquake damage, but rather, conditions of discontinuity and defects from the original construction. Further, studies have shown that NDT methods are generally unreliable in the detection of these conditions. As a result, the current recommendation is not to conduct exhaustive NDT investigations of connections in order to discover hidden damage, as was originally recommended.*
>
> *In a series of analytical investigations of the effect of moment-resisting connection damage on building behavior, it was determined that even if a large number of connections experience fracture at one beam flange to column joint, there is relatively little increase in the probability of global collapse in a future earthquake. Similarly, these investigations indicate that if both the top and bottom beam flange to column joints fracture in a large a number of connections, a very significant increase in the probability of global building collapse occurs. Therefore, to reduce the costs associated with post-earthquake inspections, with the publication of Interim Guidelines Advisory No.2 it is recommended that post-earthquake inspections initially be limited to visual inspection of the beam bottom*

flange to column joint region. If there is evidence of potential damage in this region that is not directly observable by visual means, for example, a gap between the weld backing and column flange, then supplemental investigations of this joint should be conducted using NDT. Similarly, if it is determined that fractures have occurred at the beam bottom flange joint, then inspections of that connection should be extended to encompass the entire connection including the top beam flange joint, the shear tab and column panel zone. This approach was permitted as an alternate, in the original publication of the Interim Guidelines.

~~*Some engineers have suggested an alternative approach consisting of visual-only inspections, limited to the girder bottom flange to column joint, but for a very large percentage of the total connections in the building. These bottom flange joint connections can be visually inspected with much less fireproofing removed from the framing surfaces. When significant damage is found at the exposed bottom connection, then additional fireproofing is removed to allow full exposure of the connection and inspection of the remaining surfaces. These engineers feel that by inspecting more connections, albeit to a lesser scope than recommended in these Interim Guidelines, their ability to locate the most severe occurrences of damage in a building is enhanced. These engineers use NDT assisted inspection on a very small sample of the total connections exposed to obtain an indication of the likelihood of hidden problems including damage types.*~~

~~*If properly executed, such an approach can provide sufficient information to evaluate the post-earthquake condition of a building and to make appropriate occupancy, structural repair and/or modification decisions. It is important that the visual inspector be highly trained and that visual inspections be carefully performed, preferably by a structural engineer. Casual observation may miss clues that hidden damage exists. If, as a result of the partial visual inspection, there is any reason to believe that damage exists at a connection (such as small gaps between the CJP weld backing and column face), then complete inspection of the suspected connection, in accordance with the recommendations of these Interim Guidelines should be performed. If this approach is followed, it is recommended that a significantly larger sample of connections than otherwise recommended by these Interim Guidelines, perhaps nearly all of the connections, be inspected.*~~

5.1.2 Gravity Connections

There are no modifications to the Guidelines or Commentary of Section 5.1.2 at this time.

5.1.3 Other Connection Types

There are no modifications to the Guidelines or Commentary of Section 5.1.3 at this time.

5.2 Preparation

5.2.1 Preliminary Document Review and Evaluation

5.2.1.1 Document Collection and Review

There are no modifications to the Guidelines or Commentary of Section 5.2.1.1 at this time.

5.2.1.2 Preliminary Building Walk-Through.

There are no modifications to the Guidelines or Commentary of Section 5.2.1.2 at this time.

5.2.1.3 Structural Analysis

There are no modifications to the Guidelines or Commentary of Section 5.2.1.3 at this time.

5.2.1.4 Vertical Plumbness Check

There are no modifications to the Guidelines or Commentary of Section 5.2.1.4 at this time.

5.2.2 Connection Exposure

Pre-inspection activities to expose and prepare a connection for inspection should include the local removal of suspended ceiling panels or (as applicable) local demolition of permanent ceiling finish to access the connection; and cleaning of sufficient fireproofing from the beam and column surfaces to allow visual observation of the area to be inspected. If initial inspections are to be limited to the beam bottom flange to column joint and the surrounding material, fireproofing should be removed from the connection as indicated in Figure 5-1a. Removal of fireproofing need only be sufficient to permit observation of the surfaces of base and weld metals. Wire brushing and cleaning to remove all particles of fireproofing material is not necessary unless ultrasonic testing of the joint area is to be conducted. In the event that damage is found at the bottom beam flange to column joint, then additional fireproofing should be removed, as indicated in Figure 5-1b, to expose the column panel zone, the column flange, continuity plates, beam web and flanges. The extent of the removal of fireproofing should be sufficient to allow adequate inspection of the surfaces to be inspected. Figure 5-1b suggests a pattern that will allow both visual and NDT inspection of the top and bottom beam flange to column joints, the beam web and shear connection, column panel zone and continuity plates, and column flanges in the areas of highest expected demands. The maximum extent of the removal of fireproofing need not be greater than a distance equal to the beam depth "d" into the beam span to expose evidence of any yielding.

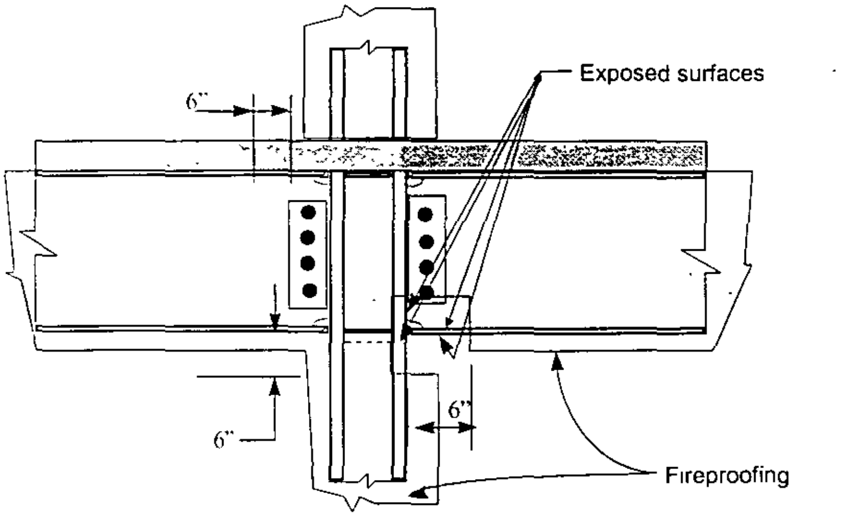


Figure 5-1a Recommended Zone for Fireproofing Removal for Initial Inspections

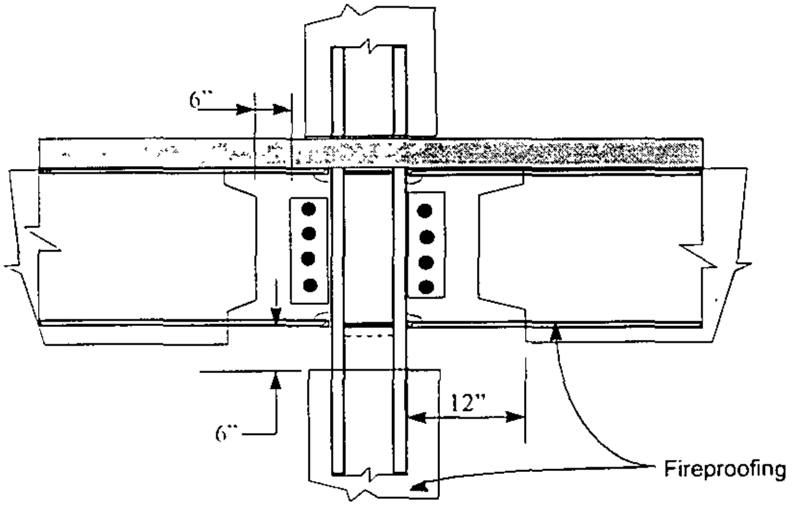


Figure 5-1b Recommended Zone for Removal of Fireproofing for Complete Inspections

Commentary If inspection is to be limited to visual observation of the surfaces of the base metal and welds, cleaning of fireproofing need only be sufficient to expose these surfaces. However, if ultrasonic testing is to be performed, the surface over which the scanning will be performed must be free ~~Cleaning of weld areas and removal~~ of mill scale and weld spatter. Such cleaning should be done with care, preferably using a power wire brush, to ensure a clean surface that does not affect the accuracy of ultrasonic testing. The resulting surface finish should be clean, free of mill scale, rust and foreign matter. The use of a chisel should be avoided to preclude scratching the steel surfaces which could be mistaken for yield lines. Sprayed-on fireproofing on WSMFs erected prior to about ~~1980~~1970 is likely to contain asbestos and should be handled according to *applicable standards for the removal of hazardous materials. Health hazards associated with asbestos were recognized by industry in the late 1960s and by 1969, most commercial production of asbestos containing materials had ceased. In April, 1973, the federal government formally prohibited the production of asbestos containing materials with the adoption of the National Emission Standards for Hazardous Air Pollutants. Allowing for shelf life of materials produced prior to that date, it should be considered possible that buildings constructed prior to 1975 contain some asbestos hazards. To preclude physical exposure to hazardous materials and working conditions in such buildings, the structural engineer should require by contractual agreement with the building owner, prior to the start of the inspection program, that the building owner deliver to the structural engineer for his/her review and files a laboratory certificate that confirms the absence of asbestos in structural steel fireproofing, local pipe insulation, ceiling tiles, and drywall joint compound.*

~~The pattern of fireproofing removal indicated in Figure 5-1 is adequate to allow visual and UT inspection of the top and bottom girder flange to column joints, the beam web and shear connection and the column panel zone. As discussed in the commentary to Section 5.1.1, some engineers prefer to initially inspect only the bottom beam flange to column joint. In such cases, the initial removal of fireproofing can be more limited than indicated in the figure. If after initial inspection, damage at a connection is suspected, then full removal, as indicated in the figure, should be performed to allow inspection of all areas of the connection~~

5.3 Inspection Program

5.3.1 Visual Inspection (VI)

There are no modifications to the Guidelines or Commentary of Section 5.3.1 at this time.

5.3.1.1 Top Flange

There are no modifications to the Guidelines or Commentary of Section 5.3.1.1 at this time.

5.3.1.2 Bottom Flange

There are no modifications to the Guidelines or Commentary of Section 5.3.1.2 at this time.

5.3.1.3 Column and Continuity Plates

There are no modifications to the Guidelines or Commentary of Section 5.3.1.3 at this time.

5.3 1 4 Beam Web Shear Connection

There are no modifications to the Guidelines or Commentary of Section 5.3.1.4 at this time.

5.3.2 Nondestructive Testing (NDT)

NDT ~~should~~ may be used to supplement the visual inspection of connections selected in accordance with the Interim Guidelines of Chapter 4. The testing agency and NDT personnel performing this work should conform to the qualifications indicated in Chapter 11 of these Interim Guidelines. The following NDT techniques~~should~~ may be used at the top and bottom of each connection, where accessible, to supplement visual inspection: These techniques should be used whenever visual inspection indicates the potential for damage that is not directly observable.

a) Magnetic particle testing (MT) of the beam flange to column flange weld surfacesmay be used to confirm the presence of suspected surface cracks based on visual evidence. Where fractures are evident from visual inspection, MT should be used to confirm thelateral extent of the fracture.~~All surfaces which were visually inspected should be tested using the magnetic particle technique~~.

   Commentary: The color of powder should be selected to achieve maximum contrast to the base and weld metal under examination. The test may be further enhanced by applying a white coating made specifically for MT or by applying penetrant developer prior to the MT examination. This background coating should be allowed to thoroughly dry before performing the MT.

b) Ultrasonic testing (UT) may be used to detect the presence of hidden fractures, where visual inspection reveals the potential for such fractures~~. of all faces at the beam flange welds and adjacent column flanges (extending at least 3 inches above and below the location of the CJP weld, along the face of the column, but not less than 1 1/2 times the column flange thickness)~~

   Commentary: The purpose of UT is to 1) locate and describe the extent of internal defects not visible on the surface and 2) to determine the extent of cracks observed visually and by MT. These guidelines recommend the use of visual inspection as the primary tool for detecting earthquake damage (See commentary to Sec 5..1.1) . UT can be a useful technique for confirmation of the presence of suspected fractures at the beam flange to column flange joints. Visual evidence that may suggest the need for such testing could include apparent separation of the base of the weld backing from the face of the column.

Requirements and acceptance criteria for NDT should be as given in AWS D1.~~1~~98 Sections 6 ~~and 8~~ Acceptance or rejection of planar weld discontinuity (cracks, slag inclusion, or lack of fusion), including root indications, should, as a minimum, be consistent with AWS Discontinuities Severity Class designations of cracks and defects per Table~~8.2~~6.2 of AWS D1.1-98 for Static

Structures. ~~Beam flange welds should be tested as "tension welds" per AWS D1.1 Table 8.15.3~~, ~~Note 3.~~ Backing bars need not be removed prior to performing UT.

> *Commentary: The value of UT for locating small discontinuities at the root of beam flange to column flange welds when the backing is left in place is not universally accepted. The reliability of this technique is particularly questionable at the center of the joint, where the beam web obscures the signal. There have been a number of reported instances of UT detected indications which were not found upon removal of the backing, and similarly, there have been reported instances of defects which were missed by UT examination but were evident upon removal of the backing. The smaller the defect, the less likely it is that UT alone will reliably detect its presence.*
>
> *Despite the potential inaccuracies of this technique, it is the only method currently available, short of removal of the backing, to find subsurface damage in the welds. It is also the most reliable method for finding lamellar problems in the column flange (type C4 and C5 damage) opposite the girder flange. Removal of weld backing at these connections results in a significant cost increase that is probably not warranted unless UT indicates widespread, significant defects and/or damage in the building.*

The proper scanning techniques, beam angle(s) and transducer sizes should be used as specified in the written UT procedure contained in the Written Practice, prepared in accordance with Section 5.3.3 of these Interim Guidelines. The acceptance standard should be that specified in the original contract documents, but in no case should it be less than the acceptance criteria of AWS D1.1~~, Chapter 8, for Statically Loaded Structures.~~

~~The base metal should be scanned with UT for cracks. Cracks which have propagated to the surface of the weld or beam and column base metal will probably have been detected by visual inspection and magnetic particle tests performed earlier. The purpose of ultrasonic testing of the base metal is to:~~

~~1. Locate and describe the extent of internal indications not apparent on the surface and,~~

~~2. Determine the extent of cracks found visually and by magnetic particle test.~~

> *Commentary: Liquid dye penetrant testing (PT) may be used where MT is precluded due to geometrical conditions or restricted access. Note that more stringent requirements for surface preparation are required for PT than MT, per AWS D1.1*
>
> ~~*If practical, NDT should be performed across the full width of the bottom beam flange joint. However, if there are no discontinuity signals from UT of*~~

~~accessible faces on one side of the bottom flange weld, obstructions on the other side of the connection need not be removed for testing of the bottom flange weld.~~

Slabs, flooring and roofing need not be removed to permit NDT of the top flange joint unless there is significant visible damage at the bottom beam flange, adjacent column flange, column web, or shear connection. Unless such damage is present, NDT of the top flange should be performed as permitted, without local removal of the diaphragms or perimeter wall obstructions.

It should be noted that UT is not 100% effective in locating discontinuities and defects in CJP beam flange to column flange welds. The ability of UT to reliably detect such defects is very dependent on the skill of the operator and the care taken in the inspection. Even under perfect conditions, it is difficult to obtain reliable readings of conditions at the center of the beam flange to column flange connection as return signals are obscured by the presence of the beam web. If backing is left in place on the welds, UT becomes even less reliable. There have been a number of reported instances in which UT indicated apparent defects, that were found not to exist upon removal of the backing. Similarly, UT has failed in some cases to locate defects that were later discovered upon removal of the backing. Additional information on UT may be found in AWS B1.10.

5.3.3 Inspector Qualification

5.3.4 Post-Earthquake Field Inspection Report

There are no modifications to the Guidelines or Commentary of Section 5.3.4 at this time.

5.3.5 Written Report

There are no modifications to the Guidelines or Commentary of Section 5.3.5 at this time.

6. POST-EARTHQUAKE REPAIR AND MODIFICATION

6.1 Scope

There are no modifications to the Guidelines or Commentary of Section 6.1 at this time.

6.2 Shoring

There are no modifications to the Guidelines or Commentary of Section 6.2 at this time.

6.3 Repair Details

There are no modifications to the Guidelines or Commentary of Section 6.3 at this time.

6.4 Preparation

There are no modifications to the Guidelines or Commentary of Section 6.4 at this time.

6.5 Execution

There are no modifications to the Guidelines or Commentary of Section 6.5 at this time.

6.6 STRUCTURAL MODIFICATION

6.6.1 Definition of Modification

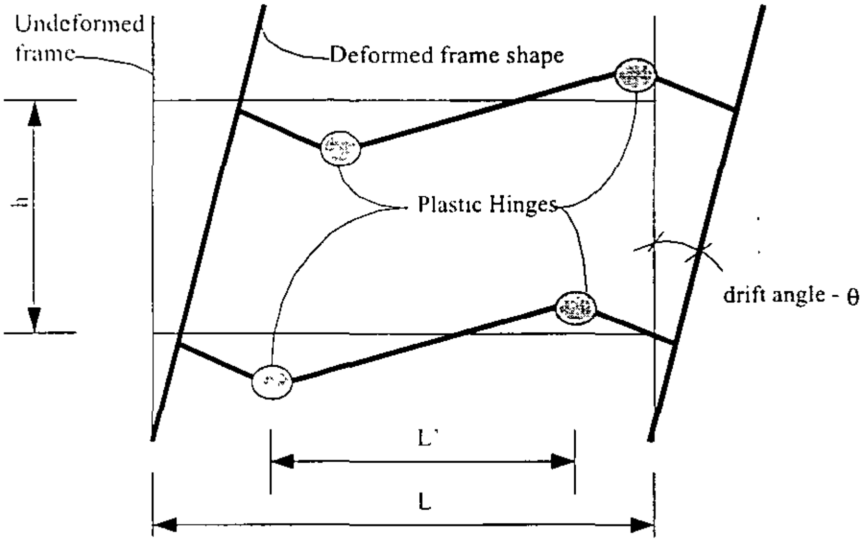
There are no modifications to the Guidelines or Commentary of Section 6.6.1 at this time.

6.6.2 Damaged vs. Undamaged Connections

There are no modifications to the Guidelines or Commentary of Section 6.6.2 at this time.

6.6.3 Criteria

Connection modification intended to permit inelastic frame behavior should be proportioned so that the required plastic deformation of the frame may be accommodated through the development of plastic hinges at pre-determined locations within the girder spans, as indicated in ~~Figure 6-12~~ Figure 6.6.3-1. Beam-column connections should be designed with sufficient strength (through the use of cover plates, haunches, side plates, etc.) to force development of the plastic hinge away from the column face. This condition may also be attained through local weakening of the beam section, at the desired location for plastic hinge formation. All elements of the connection should have adequate strength to develop the forces resulting from the formation of the plastic hinge at the predetermined location, together with forces resulting from gravity loads.



~~Figure 6-12~~ Figure 6.6.3-1 - Desired Plastic Frame Behavior

Commentary: Nonlinear deformation of frame structures is typically accommodated through the development of inelastic flexural or shear strains within discrete regions of the structure. At large inelastic strains these regions can develop into plastic hinges, which can accommodate significant concentrated rotations at constant (or nearly constant) load through yielding at tensile fibers and buckling at compressive fibers. If a sufficient number of plastic hinges develop in a frame, a mechanism is formed and the frame can deform laterally in a plastic manner. This behavior is accompanied by significant energy dissipation, particularly if a number of members are involved in the plastic behavior, as well as substantial local damage to the highly strained elements. The formation of hinges in columns, as opposed to beams, is undesirable, as this results in the formation of weak story mechanisms with relatively few elements participating, and consequently little energy dissipation occurring. In addition, such mechanisms also result in local damage to critical gravity load bearing elements.

The prescriptive connection contained in the UBC and NEHRP Recommended Provisions prior to the Northridge Earthquake was based on the assumed development of plastic hinge zones within the beams ~~at~~ adjacent to the face of the column, or within the column panel zone itself. If the plastic hinge develops in the column panel zone, the resulting column deformation results in very large secondary stresses on the beam flange to column flange joint, a condition which can contribute to brittle failure. If the plastic hinge forms in the beam, at the face of the column, this can result in very large ~~through thickness strain demands on~~

~~the column flange material and large inelastic strain demands on the weld metal and surrounding heat affected zones~~ stress and strain demands on the welded beam flange to column flange joint. These conditions can also lead to brittle joint failure. Although ongoing research may reveal conditions of material properties, design and detailing configurations that permit connections with yielding occurring at the column face to perform reliably, for the present, it is recommended ~~In order to achieve more reliable performance, it is recommended~~ that the connection of the beam to the column be modified to be sufficiently strong to force the inelastic action (plastic hinge) away from the column face. Plastic hinges in steel beams have finite length, typically on the order of half the beam depth. Therefore, the location for the plastic hinge should be shifted at least that distance away from the face of the column. When this is done, the flexural demands on the columns are increased. Care must be taken to assure that weak column conditions are not inadvertently created by local strengthening of the connections

It should be noted that connection modifications of the type described above, while believed to be effective in preventing brittle connection fractures, will not prevent structural damage from occurring. Brittle connection fractures are undesirable because they result in a substantial reduction in the lateral-force-resisting strength of the structure which, in extreme cases, can result in instability and collapse. Connections modified as described in these Interim Guidelines should experience many fewer such brittle fractures than unmodified connections. However, the formation of a plastic hinge within the span of a beam is not a completely benign event Beams which have formed such hinges may exhibit large buckling and yielding deformation, damage which typically must be repaired. The cost of such repairs could be comparable to the costs incurred in repairing fracture damage experienced in the Northridge Earthquake. The primary difference is that life safety protection will be significantly enhanced and most structures that have experienced such plastic deformation damage should continue to be safe for occupancy while repairs are made.

If the types of damage described above are unacceptable for a given building, then alternative methods of structural modification should be considered that will reduce the plastic deformation demands on the structure during a strong earthquake. Appropriate methods of achieving such goals include the installation of supplemental braced frames, energy dissipation systems, and similar systematic modifications of the building's basic lateral force resisting system.

It is important to recognize that in frames with relatively short bays, the flexural hinging indicated in Figure 6.6.3-1 may not be able to form. If the effective flexural length (L' in the figure) of beams in a frame becomes too short, then the beams or girders will yield in shear before zones of flexural plasticity *can form, resulting in an inelastic behavior that is more like that of an eccentrically braced frame than that of a moment frame. This behavior may inadvertently occur in frames in which relatively large strengthened connections, such as haunches, cover plates or side plates have been used on beams with relatively short spans. This behavior is illustrated in Figure 6.6.3-2.*

The guidelines contained in this section are intended to address the design of flexurally dominated moment resisting frames. When utilizing these guidelines, it is important to confirm that the configuration of the structure is such that the presumed flexural hinging can actually occur. It is possible that shear yielding of frame beams, such as that schematically illustrated in Figure 6.6.3-2 may be a desirable behavior mode. However, to date, there has not been enough research conducted into the behavior of such frames to develop recommended design guidelines. If modifications to an existing frame result in such a configuration designers should consider referring to the code requirements for eccentrically braced frames. Particular care should be taken to brace the shear link of such beams against lateral-torsional buckling and also to adequately stiffen the webs to avoid local buckling following shear plastification.

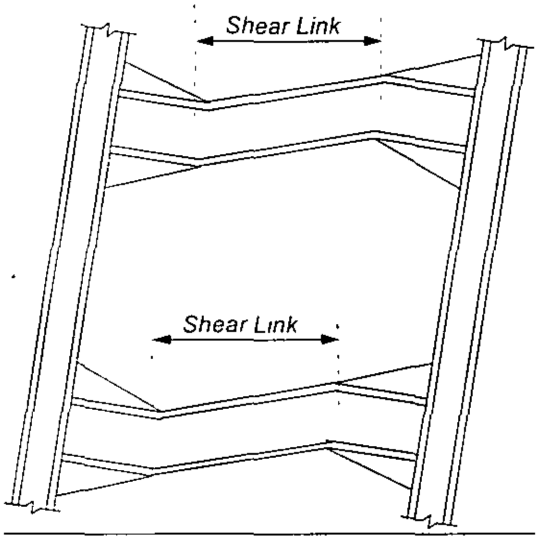


Figure 6.6.3-2 Shear Yielding Dominated Behavior of Short Bay Frames

6.6.4 Strength and Stiffness

6.6.4.1 Strength

When these Interim Guidelines require determination of the strength of a framing element or component, this shall be calculated in accordance with the criteria contained in *UBC-94*, Section

2211.4.2 {*NEHRP-91* Section 10.2, except that the factor ϕ should be taken as 1.0}, restated as follows:

2211.4.1 Member strength. Where this section requires that the strength of the member be developed, the following shall be used:

Flexure	$M_s = Z F_y$
Shear	$V_s = 0.55 F_y d t$
Axial compression	$P_{sc} = 1.7 F_a A$
Axial tension	$P_{st} = F_y A$
Connectors	
Full Penetration welds	$F_y A$
Partial Penetration welds	1.7 allowable (see commentary)
Bolts and fillet welds	1.7 allowable

Alternatively, the criteria contained in the 1997 edition of the*AISC Seismic Provisions for Structural Steel Buildings* (AISC, 1997) may be used.

Commentary: At the time the Interim Guidelines were first published, they were based on the 1994 edition of the Uniform Building Code and the 1994 edition of the NEHRP Provisions. In the time since that initial publication, more recent editions of both documents have been published, and codes based on these documents have been adopted by some jurisdictions. In addition, the American Institute of Steel Construction has adopted a major revision to its Seismic Provisions for Structural Steel Buildings (AISC Seismic Provisions), largely incorporating, with some modification, the recommendations contained in the Interim Guidelines. This updated edition of the AISC Seismic Provisions has been incorporated by reference into the 1997 edition of the NEHRP Provisions and has also been adopted by some jurisdictions as an amendment to the model building codes. Structural upgrades designed to comply with the requirements of the 1997 AISC Seismic Provisions may be deemed to comply with the intent of these Interim Guidelines. Where reference is made herein to the requirements of the 1994 Uniform Building Code or 1994 NERHP Provisions, the parallel provisions of the 1997 editions may be used instead, and should be used in those jurisdictions that have adopted codes based on these updated standards.

Partial penetration welds are not recommended for tension applications in critical connections resisting seismic induced stresses. The geometry of partial penetration welds creates a notch-like condition that can initiate brittle fracture under conditions of high tensile strain.

Many WSMF structures are constructed with concrete floor slabs that are provided with positive shear attachment between the slab and the top flanges of the girders of the moment-resisting frames. Although not generally accounted for in the design of the frames, the resulting composite action can increase the *effective strength of the girder significantly, particularly at sections where curvature of the girder places the top flange into compression. Although this effect is directly accounted for in the design of composite systems, it is typically neglected in the design of systems classified as moment resisting steel frames. The increased girder flexural strength caused by this composite action can result in a number of effects including the unintentional creation of weak column - strong beam and weak panel zone conditions. In addition, this composite effect has the potential to reduce the effectiveness of reduced section or "dog-bone" type connection assemblies. Unfortunately, very little laboratory testing of large scale connection assemblies with slabs in place has been performed to date and as a result, these effects are not well quantified. In keeping with typical contemporary design practice, the design formulae provided in these Guidelines neglect the strengthening effects of composite action. Designers should, however, be alert to the fact that these composite effects do exist. Similar, and perhaps more severe, effects may also exist where steel beams support masonry or concrete walls.*

6.6.4.2 Stiffness

Calculation of frame stiffness for the purpose of determining interstory drift under the influence of the design lateral forces should be based on the properties of the bare steel frame, neglecting the effects of composite action with floor slabs. The stiffening effects of connection reinforcements (e.g.: haunches, side plates, etc.) may be considered in the calculation of overall frame stiffness and drift demands. When reduced beam section connections are utilizedthe reduction in overall frame stiffness, due to local reductions in girder cross section, should be considered

Commentary: For design purposes, frame stiffness is typically calculated considering only the behavior of the bare frame, neglecting the stiffening effects of slabs, gravity framing, and architectural elements such as walls. The resulting calculation of building stiffness and period typically underestimates the actual properties, substantially. Although this approach can result in unconservative estimates of design force levels, it typically produces conservative estimates of interstory drift demands. Since the design of most moment-resisting frames are controlled by considerations of drift, this approach is considered preferable to methods that would have the potential to over-estimate building stiffness. Also, many of the elements that provide additional stiffness may be subject to rapid degradation under severe cyclic lateral deformation, so that the bare frame stiffness may provide a reasonable estimate of the effective stiffness under long duration ground shaking response

Notwithstanding the above, designers should be alert to the fact that unintentional stiffness introduced by walls and other non-structural elements can *significantly alter the behavior of the structure in response to ground shaking. Of particular concern, if these elements are not uniformly distributed throughout the structure, or isolated from its response, they can cause soft stories and torsional irregularities, conditions known to result in poor behavior.*

6.6.5 Plastic Rotation Capacity

The plastic rotation capacity of modified connections should reflect realistic estimates of the required level of plastic rotation demand. In the absence of detailed calculations of rotation demand, connections should be shown to be capable of developing a minimum plastic rotation capacity on the order of 0.025 to 0.030 radian. The demand may be lower when braced frames, supplemental damping, base isolation, or other elements are introduced into the moment frame system, to control its lateral deformation; when the design ground motion is relatively low in the range of predominant periods for the structure; and when the frame is sufficiently strongand stiff.

As used in these Guidelines, plastic rotation is defined as the plastic chordrotation angle. The plastic chord rotation angle is calculated using the rotated coordinate system shown in Fig. 6.6.5-1, as the plastic deflection of the beam or girder, at the point of inflection (usually at thecenter of its span,) Δ_{CL}, divided by the distance between the center of the beam span and the centerline of the panel zone of the beam column connection, L_{CL}. This convention is illustrated in Figure6.6.5-1.

It is important to note that this definition of plastic rotation is somewhat different than the plastic rotation that would actually occur within a discrete plastic hinge in a frame model similar to that shown in Figure 6.6.3-1. These two quantities are related to each other, however, and if one of them is known, the other may be calculated fromEq. 6.6.5-1.

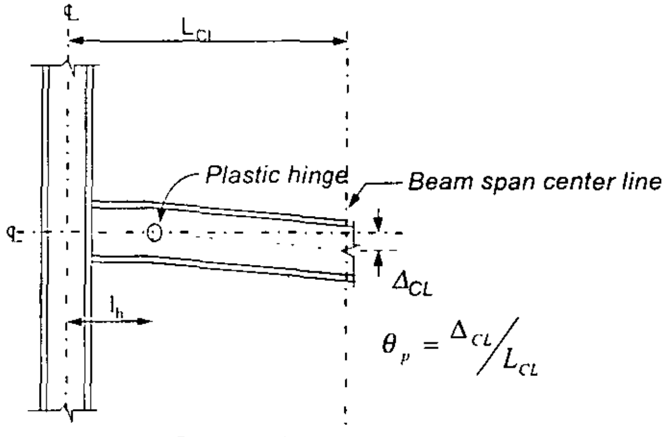


Figure 6.6.5-1 Calculation of Plastic Rotation Angle

$$\theta_p = \theta_{ph} \frac{\left(L_{CL} - l_h\right)}{L_{CL}} \tag{6.6.5-1}$$

where, θ_p is the plastic chord angle rotation, as used in these Guidelines
θ_{ph} is the plastic rotation, at the location of a discrete hinge
L_{CL} is the distance from the center of the beam span to the center of the beam-column assembly panel zone
l_h is the assumed location of the discrete plastic hinge relative to the center of the beam-column assembly panel zone

If calculations are performed to determine the required connection plastic rotation capacity, the capacity should be taken somewhat greater than the calculated deformation demand, due to the high variability and uncertainty inherent in predictions of inelastic seismic response. Until better guidelines become available, a required plastic rotation capacity on the order of 0.005 radians greater than the demand calculated for the design basis earthquake (or if greater conservatism is desired - the maximum ~~capable~~ considered earthquake) is recommended. Rotation demand calculations should consider the effect of plastic hinge location within the beam span, as indicated in ~~Figure 6-12~~ Figure 6.6.3-1, on plastic rotation demand. Calculations should be performed to the same level of detail specified for nonlinear dynamic analysis for base isolated structures in *UBC-94* Section 1655 {*NEHRP-94* Section 2.6.4.4}. Ground motion time histories utilized for these nonlinear analyses should satisfy the scaling requirements of *UBC-94* Section 1655.4.2 {*NEHRP-94* Section 2.6.4.4} except that instead of the base isolated period, T_I, the structure period, T, calculated in accordance with *UBC-94* Section 1628 {*NEHRP-94* Section 2.3.3.1} should be used.

> *Commentary: When the Interim Guidelines were first published, the plastic rotation was defined as that rotation that would occur at a discrete plastic hinge, similar to the definition of θ_{ph}, in Eq. 6.6.5-1, above. In subsequent testing of prototype connection assemblies, it was found that it is often very difficult to determine the value of this rotation parameter from test data, since actual plastic hinges do not occur at discrete points in the assembly and because some amount of plasticity also occurs in the panel zone of many assemblies. The plastic chord angle rotation, introduced in Interim Guidelines Advisory No. 1, may more readily be obtained from test data and also more closely relates to the drift experienced by a frame during earthquake response*
>
> *Traditionally, structural engineers have calculated demand in moment frames by sizing the members for strength and drift using code forces (either equivalent static or reduced dynamic forces) and then "developing the strength of the members." Since 1988, "developing the strength" has been accomplished by prescriptive means. It was assumed that the prescribed connections would be strong enough so that the girder would yield (in bending), or the panel zone*

would yield (in shear) in a nearly perfectly plastic manner producing the plastic rotations necessary to dissipate the energy of the earthquake. It is now known that the prescriptive connection is often incapable of behaving in this manner.

In the 1994 Northridge earthquake, many moment-frame connections fractured with little evidence of plastic hinging of the girders or yielding of the column panel zones. Testing of moment frame connections both prior to and subsequent to the earthquake suggests that the standard welded flange-bolted web connection is unable to reliably provide plastic rotations beyond about 0.005 radian for all ranges of girder depths and often fails below that level. Thus, for frames designed for code forces and for the code drift limits, new connection configurations must be developed to reliably accommodate such rotation without brittle fracture.

In order to develop reasonable estimates of the plastic rotation demands on a frame's connections, it is necessary to perform inelastic time history analyses. For regular structures, approximations of the plastic rotation demands can be obtained from linear elastic analyses. Analytical research (Newmark and Hall - 1982) suggests that for structures having the dynamic characteristics of most WSMF buildings, and for the ground motions typical of western US earthquakes, the total frame deflections obtained from an unreduced (no R or R_w factor) dynamic analysis provide an approximate estimate of those which would be experienced by the inelastic structure. For the typical spectra contained in the building code, this would indicate expected drift ratios on the order of 1%. The drift demands in a real structure, responding inelastically, tend to concentrate in a few stories, rather than being uniformly distributed throughout the structure's height. Therefore, it is reasonable to expect typical drift demands in individual stories on the order of 1.5% to 2% of the story height. As a rough approximation, the drift demand may be equated to the joint rotation demand, yielding expected rotation demands on the order of perhaps 2%. Since there is considerable variation in ground motion intensity and spectra, as well as the inelastic response of buildings to these ground motions, conservatism in selection of an appropriate connection rotation demand is warranted.

In recent testing of large scale subassemblies incorporating modified connection details, conducted by SAC and others, when the connection design was able to achieve a plastic rotation demand of 0.025 radians or more for several cycles, the ultimate failure of the subassembly generally did not occur in the connection, but rather in the members themselves. Therefore, the stated connection capacity criteria would appear to result in connections capable of providing reliable performance.

It should be noted that the connection assembly capacity criteria for the modification of existing buildings, recommended by these Interim Guidelines, is *somewhat reduced compared to that recommended for new buildings (Chapter 7). This is typical of approaches normally taken for existing structures. For new buildings, these Interim Guidelines discourage building-specific calculation of required plastic rotation capacity for connections and instead, encourage the development of highly ductile connection designs. For existing buildings, such an approach may lead to modification designs that are excessively costly, as well as the modification of structures which do not require such modification. Consequently, an approach which permits the development of semi-ductile connection designs, with sufficient plastic rotation capacity to withstand the expected demands from a design earthquake is adopted. It should be understood that buildings modified to this reduced criteria will not have the same reliability as new buildings, designed in accordance with the recommendations of Chapter 7. The criteria of Chapter 7 could be applied to existing buildings, if superior reliability is desired.*

When performing inelastic frame analysis, in order to determine the required connection plastic rotation capacity, it is important to accurately account for the locations at which the plastic hinges will occur. Simplified models, which represent the hinge as occurring at the face of the column, may~~will~~ underestimate the plastic rotation demand. This problem becomes more severe as the column spacing, L, becomes shorter and the distance between plastic hinges, L,' a greater portion of the total beam span. Eq. 6.6.5-1 may be used to convert calculated values of plastic rotation at a hinge remotely located from the column, to the chord angle rotation, used for the definition of acceptance criteria contained in these Guidelines. In extreme cases, the girder will not form plastic hinges at all, but instead, will develop a shear yield, similar to an eccentric braced frame.

6.6.6 Connection Qualification and Design

Modified girder-column connections may be qualified by testing or designed using calculations. Qualification by testing is the preferred approach. Preliminary designs of connections to be qualified by test may be obtained using the calculation procedures of Section 6.6.6.3 The procedures of that section may also be used to calibrate previous tests of similar connection configurations to slightly different applications, by extrapolation. Extrapolation of test results should be limited to connections of elements having similar geometries and material specifications as the tested connections Designs based on calculation alone should be subject to qualified independent third party review.

Commentary: Because of the cost of testing, use of calculations for interpolation or extrapolation of test results is desirable. How much extrapolation should be accepted is a difficult decision. As additional testing is done, more information may be available on what constitutes "conservative" testing conditions, thereby *allowing easier decisions relative to extrapolating tests to actual conditions which are likely to be less demanding than the tests. For example, it is hypothesized that connections of shallower, thinner flanged members are likely to be more reliable than similar connections consisting of deeper, thicker flanged members. Thus, it may be possible to test the largest assemblages of similar details and extrapolate to the smaller member sizes? - at least within comparable member group families. However, there is evidence to suggest that extrapolation of test results to assemblies using members of reduced size is not always conservative. In a recent series of tests of cover plated connections, conducted at the University of California at San Diego, a connection assembly that produced acceptable results for one family of beam sizes, W24, did not behave acceptably when the beam depth was reduced significantly to W18. In that project, the change in relative flexibilities of the members and connection elements resulted in a shift in the basic behavior of the assembly and initiation of a failure mode that was not observed in the specimens with larger member sizes. In order to minimize the possibility of such occurrences, when extrapolation of test results is performed, it should be done with a basic understanding of the behavior of the assembly, and the likely effects of changes to the assembly configuration on this behavior. Test results should not be extrapolated to assembly configurations that are expected to behave differently than the tested configuration. Extrapolation or interpolation of results with differences in welding procedures, details or material properties is even more difficult*

6.6.6 1 Qualification Test Protocol

There are no modifications to the Guidelines or Commentary of Section 6.6.6.1 at this time.

6.6 6.2 Acceptance Criteria

The minimum acceptance criteria for connection qualification for specimens tested in accordance with these Interim Guidelines should be as follows:

a) The connection should develop beam plastic rotations as indicated in Section 6.6.5, for at least one complete cycle.

b) The connection should develop a minimum strength equal to 80% of the plastic strength of the girder, calculated using minimum specified yield strength F_y throughout the loading history required to achieve the required plastic rotation capacity, as indicated in a), above

c) The connection should exhibit ductile behavior throughout the loading history. A specimen that exhibits a brittle limit state (e.g. complete flange fracture, column cracking, through-thickness failures of the column flange, fractures in welds subject to tension, shear tab cracking, etc.) prior to reaching the required plastic rotation shall be considered unsuccessful.

d) Throughout the loading history, until the required plastic rotation isachieved, the connection should be judged capable of supporting dead and live loads required by the building code. In those specimens where axial load is applied during the testing, the specimen should be capable of supporting the applied load throughout the loading history.

The evaluation of the test specimen's performance should consistently reflect the relevant limit states. For example, the maximum reported moment and the moment at the maximum plastic rotation are unlikely to be the same. It would be inappropriate to evaluate the connection using the maximum moment and the maximum plastic rotation in a way that implies that they occurred simultaneously. In a similar fashion, the maximum demand on the connection should be evaluated using the maximum moment, not the moment at the maximum plastic rotation nless the behavior of the connection indicated that this limit state produced a more critical condition in the connection.

Commentary: Many connection configurations will be able to withstand plastic rotations on the order of 0.025 radians or more, but will have sustained significant damage and degradation of stiffness and strength in achieving this deformation. The intent of the acceptance criteria presented in this Section is to assure that when connections experience the required plastic rotation demand, they will still have significant remaining ability to participate in the structure's lateral load resisting system.

In evaluating the performance of specimens during testing, it is important to distinguish between brittle behavior and ductile behavior. It is not uncommon for small cracks to develop in specimens after relatively few cycles of inelastic deformation. In some cases these initial cracks will rapidly lead to ultimate failure of the specimen and in other cases they will remain stable, perhaps growing slowly with repeated cycles, and may or may not participate in the ultimate failure mode. The development of minor cracks in a specimen, prior to achievement of the target plastic rotation demand should not be cause for rejection of the design if the cracks remain stable during repeated cycling. Similarly, the occurrence of brittle fracture at inelastic rotations significantly in excess of the target plastic rotation should not be cause for rejection of the design.

6.6 6.3 Calculations

There are no modifications to the Guidelines or Commentary of Section 6.6.6.3 at this time.

6.6.6.3.1 Material Strength Properties

In the absence of project specific material property information (for example, mill test reports), the values listed in ~~Table 6-3~~ Table 6.6.6.3.1-1 should be used to determine the strength of steel shape and plate for purposes of calculation. The permissible strength for weld metal should be taken in accordance with the building code.

~~Table 6-3~~Table 6.6.6.3.1-1 - Properties for Use in Connection Modification Design

Material	F_y (ksi)	$F_{y\,m}$ (ksi)	F_u (ksi)
A36 Beam	36	^{1}	^{1}
Dual Certified Beam			
Axial, Flexural	50		65 min.
Shape Group 1		55^2	
Shape Group 2		58^2	
Shape Group 3		57^2	
Shape Group 4		54^2	
Through-Thickness	-	-	Note 3
A572 Column/Beam			
Axial, Flexural	50		65 min.
Shape Group 1		58^2	
Shape Group 2		58^2	
Shape Group 3		57^2	
Shape Group 4		57^2	
Shape Group 5		55^2	
Through-Thickness	-	-	Note 3
A992 Structural Shape^1	Use same values as for A572, Gr. 50		

Notes.

1. See Commentary
2. Based on coupons from web. For thick flanges, the $F_{y\ flange}$ is approximately 0.95 $F_{y\ web}$.
3. See Commentary

Commentary: Table 6-3. Note 1 - The material properties for steel nominally designated on the construction documents as ASTM A36 can be highly variable and in recent years, steel meeting the specified requirements for both ASTM A36 and A572 has routinely been incorporated in projects calling for A36 steel. Consequently, unless project specific data is available to indicate the actual strength of material incorporated into the project, the properties for ASTM A572 steel should be assumed when ASTM A36 is indicated on the drawings, and the assumption of a higher yield stress results in a more severe design condition.

The ASTM A992 specification was specifically developed by the steel industry in response to expressed concerns of the design community with regard to the permissible variation in chemistry and mechanical properties of structural steel under the A36 and A572 specifications. This new specification, which was adopted in late 1998, is very similar to ASTM A572, except that it includes somewhat more restrictive limits on chemistry and on the permissible variation in *yield and ultimate tensile stress, as well as the ratio of yield to tensile strength. At this time, no statistical data base is available to estimate the actual distribution of properties of material produced to this specification. However, the properties are likely to be very similar, albeit with less statistical scatter, to those of material recently produced under ASTM A572, Grade 50.*

~~Table 6-3~~*Table 6.6.6.3-1, Note 3 - In the period immediately following the Northridge earthquake, the Seismology Committee of the Structural Engineers Association of California and the International Conference of Building Officials issued Interim Recommendation No. 2 (SEAOC-1995) to provide guidance on the design of moment resisting steel frame connections. Interim Recommendation No. 2 included a recommendation that the through-thickness stress demand on column flanges be limited to a value of 40 ksi, applied to the projected area of beam flange attachment. This value was selected somewhat arbitrarily, to ensure that through-thickness yielding did not initiate in the column flanges of moment-resisting connections and because it was consistent with the successful tests of assemblies with cover plates conducted at the University of Texas at Austin (Engelhardt and Sabol - 1994), rather than being the result of a demonstrated through-thickness capacity of typical column flange material. Despite the somewhat arbitrary nature of the selection of this value, its use often controls the overall design of a connection assembly including the selection of cover plate thickness, haunch depth, and similar parameters.*

It would seem to be important to prevent the inelastic behavior of connections from being controlled by through-thickness yielding of the column flanges. This is because it would be necessary to develop very large local ductilities in the column flange material in order to accommodate even modest plastic rotation demands on the assembly. However, extensive investigation of the through-thickness behavior of column flanges in a "T" joint configuration reveals that neither yielding, nor through-thickness failure are likely to occur in these connections. Barsom and Korvink (1997) conducted a statistical survey of available data on the tensile strength of rolled shape material in the through-thickness direction. These tests were generally conducted on small diameter coupons, extracted from flange material of heavy shapes. The data indicates that both the yield stress and ultimate tensile strength of this material in the through-thickness direction is comparable to that of the material in the direction parallel to rolling. However, it does indicate somewhat greater scatter, with a number of reported values where the through-thickness strength was higher, as well as lower than that in the longitudinal direction. Review of this data indicates with high confidence that for small diameter coupons, the yield and ultimate tensile values of the material in a through-thickness direction will exceed 90% and 80% respectively of the comparable values in the longitudinal direction. ~~*theThe causes for through-thickness failures of column flanges (types C2, C4, and C5), observed*~~

~~*both in buildings damaged by the Northridge Earthquake and in some test specimens, are not well understood. They are thought to be a function of the metallurgy and "purity," of the steel: conditions of loading including the presence of axial load and rate of loading application; conditions of tri-axial restraint; conditions of local hardening and embrittlement within the weld's heat affected zone; stress concentrations induced by the presence of backing bars and defects at the root of beam flange to column flange welds; and by the relationship of the connection components as they may affect flange bending stresses and flange curvature induced by panel zone yielding. Given the many complex factors which can affect the through-thickness strength of the column flange, determination of a reliable basis upon which to set permissible design stresses will require significant research. Such research is currently being conducted under the SAC phase II program.*~~

While this statistical distribution suggests the likelihood that the through-thickness strength of column flanges could be less than the flexural strength of attached beam elements, testing of more than 40 specimens at Lehigh University indicates that this is not the case. In these tests, high strength plates, representing beam flanges and having a yield strength of 100 ksi were welded to the face of A572, Grade 50 and A913, Grade 50 column shapes, to simulate the portion of a beam-column assembly at the beam flange. These specimens were placed in a universal testing machine and loaded to produce high through-thickness tensile stresses in the column flange material. The tests simulated a wide range of conditions, representing different weld metals as well and also to include eccentrically applied loading. In 40 of 41 specimens tested, the assembly strength was limited by tensile failure of the high strength beam flange plate as opposed to the column flange material. In the one failure that occurred within the column flange material, fracture initiated at the root of a low-toughness weld, at root defects that were intentionally introduced to initiate such a fracture.

The behavior illustrated by this test series is consistent with mechanics of materials theory. At the joint of the beam flange to column flange, the material is very highly restrained. As a result of this, both the yield strength and ultimate tensile strength of the material in this region is significantly elevated. Under these conditions, failure is unlikely to occur unless a large flaw is present that can lead to unstable crack propagation and brittle fracture. In light of this evidence, Interim Guidelines Advisory No. 2 deletes any requirement for evaluation of through-thickness flange stress in columns.

~~*Interim Recommendation No. 2 (SEAOC-1995) included a value of 40 ksi, applied to the projected area of beam flange attachment, for the through-thickness strength to be used in calculations. This value was selected because it was consistent with the successful tests of cover plated assemblies conducted at*~~

~~*the University of Texas at Austin (Engelhardt and Sabol - 1994). However, because of the probable influence of all the factors noted above, this value can only be considered to reflect the specific conditions of those tests and specimens.*~~

~~*Although reduced stresses at the column face produced acceptable results in the University of Texas tests, the key to that success was more likely the result of forcing the plastic hinge away from the column than reduction of the through-thickness stress by the cover plates. Reduction of through thickness column flange stress to ever lower levels by the use of thicker cover plates is not recommended, since such cover plates will result in ever higher forces on the face of the column flange as well as larger weldments with potential for enlarged heat affected zones, higher residual stresses and conditions of restraint.*~~

~~*Since the initial publication of the Interim Guidelines, a significant number of tests have been performed on reduced beam section connections (See section 7.5.3), most of which employed beam flanges which were welded directly to the column flanges using improved welding techniques, but without reinforcement plates. No through thickness failures occurred in these tests despite the fact that calculated through thickness stresses at the root of the beam flange to column flange joint ranged as high as 58 ksi. The successful performance of these welded joints is most probably due to the shifting of the yield area of the assembly away from the column flange and into the beam span. Based on the indications of the above described tests, and noting the undesirability of over reinforcing connections, it is now suggested that a maximum through thickness stress of 0.9F_{yc} may be appropriate for use with connections that shift the hinging away from the column face. Notwithstanding this recommendation, engineers are still cautioned to carefully consider the through thickness issue when these other previously listed conditions which are thought to be involved in this type of failure are prevalent.*~~

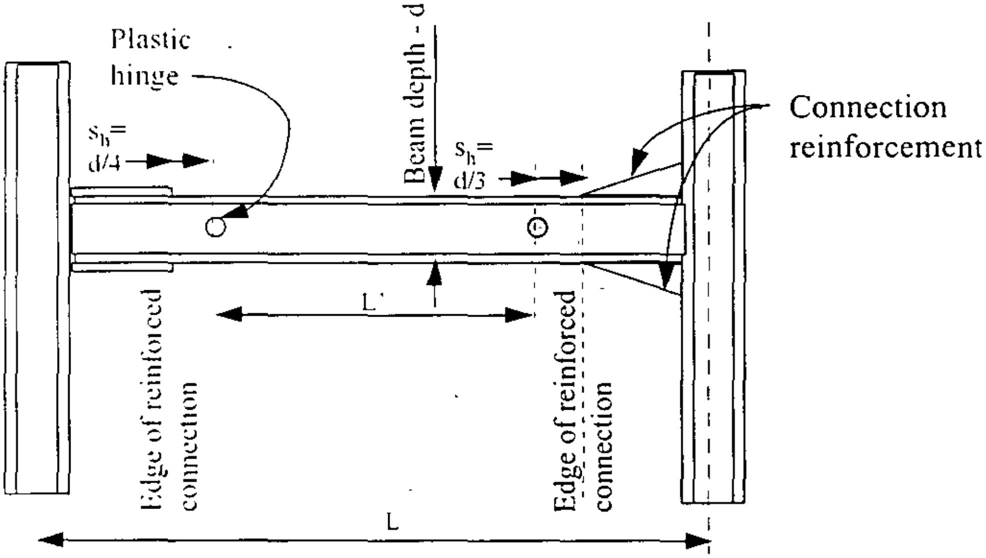
~~*Notwithstanding all of the above, successful tests using cover plates and other measures of moving hinges (and coincidentally reducing through thickness stress) continue to be performed. In the interim, structural engineers choosing to utilize connections relying on through thickness strength should recognize that despite the successful testing, connections relying on through thickness strength can not be considered to be fully reliable until the influence of the other parameters discussed above can be fully understood. A high amount of structural redundancy is recommended for frames employing connections which rely on through thickness strength of the column flange.*~~

6.6.6.3.2 Determine Plastic Hinge Location

The desired location for the formation of plastic hinges should be determined as a basic parameter for the calculations. For beams with gravity loads representing a small portion of the total flexural demand, the location of the plastic hinge may be assumed to occur as indicated in Table 6.6.6.3.2-1 and illustrated in Figure 6.6.6.3.2-1, ~~at a distance equal to 1/3 of the beam depth from the edge of the reinforced connection (or start of the weakened beam section),~~ unless specific test data for the connection indicates that a different value is appropriate~~. Refer to Figure 6-13.~~

Table 6.6.6.3.2-1 Plastic Hinge Location - Strengthened Connections

Connection Type	Reference Section	Hinge Location "s_h"
Cover plates	Sect. 7.9.1	d/4 beyond end of cover plates
Haunches	Sect. 7.9.3, 7.9.4	d/3 beyond toe of haunch
Vertical Ribs	Sect. 7.9.2	d/3 beyond toe of ribs



~~Figure 6-13~~ Figure 6.6.6.3.2-1 - Location of Plastic Hinge

Commentary: The suggested locations for the plastic hinge~~, at a distance d/3 away from the end of the reinforced section~~ indicated in Table 6.6.6.3.2-1 and Figure 6.6.6.3.2-1 are ~~is~~ based on the observed behavior of test specimens, with no significant gravity load present. If significant gravity load is present, this can shift the locations of the plastic hinges, and in the extreme case, even change the form of the collapse mechanism. If flexural demand on the girder due to gravity load is less than about 30% of the girder plastic capacity, this effect can safely be neglected, and the plastic hinge locations taken as indicated. If gravity demands significantly exceed this level then plastic analysis of the girder should be *performed to determine the appropriate hinge locations. Note that in zones of high seismicity (UBC Zones 3 and 4, and NEHRP Map Areas 6 and 7) gravity loading on the girders of earthquake resisting frames typically has a very small effect.*

6.6.6.3.3 Determine Probable Plastic Moment at Hinges

The probable value of the plastic moment, M_{pr}, at the location of the plastic hinges should be determined from the equation:

~~$M_{pr} = 0.95\alpha Z_b F_{ya}$ (6-1)~~

$$M_{pr} = 1.1 Z_b F_{ya} \qquad (6.6.6.3.3\text{-}1)$$

where, ~~α is a coefficient that accounts for the effects of strain hardening and modeling uncertainty, taken as:~~

~~1.1 when qualification testing is performed or calculations are correlated with previous qualification testing~~

~~1.3 when design is based on calculations, alone.~~

F_{ya} is the actual yield stress of the material, as identified from mill test reports. Where mill test data for the project is not traceable to specific framing elements, the average of mill test data for the project for the given shape may be used. When mill test data for the project is not available, the value of F_{ym}, from ~~table 6-3~~Table 6.6.6.3-1 may be used.

Z_b is the plastic modulus of the section

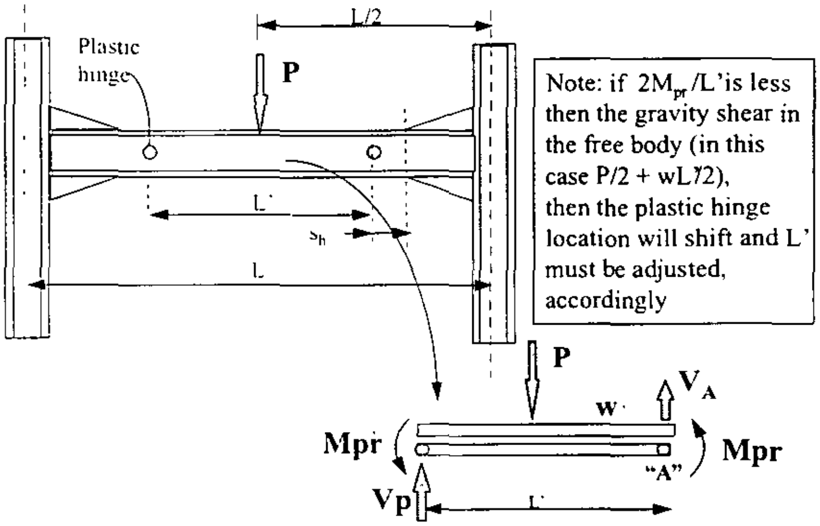
Commentary: The 1.1~~0.95~~ factor, in equation 6.6.6.3.3-1, is used to ~~adjust~~ account for two effects. First, it is intended to account for the typical difference between the yield stress in the beam web, where coupons for mill certification tests are normally extracted, and~~to~~ the value in the beam flange. Beam flanges, being comprised of thicker material, typically have somewhat lower yield strengths than do beam web material. Second, it is intended to~~The factor of 1.1 recommended to~~ account for strain hardening, or other sources of strength above yield, and agrees fairly well with available test results. It should be noted that the 1.1 factor could underestimate the over-strength where significant flange buckling does not act as the gradual limit on the connection. Nevertheless, the 1.1 factor seems a reasonable expectation of over-strength considering the complexities involved.

Connection designs that result in excessive strength in the girder connection relative to the column or excessive demands on the column panel zone are not expected to produce superior performance. There is a careful balance that must be maintained between developing connections that provide for an appropriate allowance for girder overstrength and those that arbitrarily increase connection demand in the quest for a "conservative" connection design. The factors suggested above were chosen in an attempt to achieve this balance, and arbitrary increases in these values are not recommended.

When the Interim Guidelines were first published, Eq. 6.6.6.3.3-1 included a coefficient, α, intended to account both for the effects of strain hardening and also for modeling uncertainty when connection designs were based on calculations as opposed to a specific program of qualification testing. The intent of this modeling uncertainty factor was twofold: to provide additional conservatism in the design when specific test data for a representative connection was not available, and also as an inducement to encourage projects to undertake connection qualification testing programs. After the Interim Guidelines had been in use for some time, it became apparent that this approach was not an effective inducement for projects to perform qualification testing, and also that the use of an overly large value for the α coefficient often resulted in excessively large connection reinforcing elements (cover plates, e.g.) and other design features that did not appear conducive to good connection behavior. Consequently, it was decided to remove this modeling uncertainty factor from the calculation of the probable strength of an assembly

6.6.6.3.4 Determine Beam Shear

The shear in the beam at the location of the plastic hinge should be determined. A free body diagram of that portion of the beam located between plastic hinges is a useful tool for obtaining the shear at each plastic hinge. ~~Figure 6-14~~Figure 6.6.3.4-1 provides an example of such a calculation.



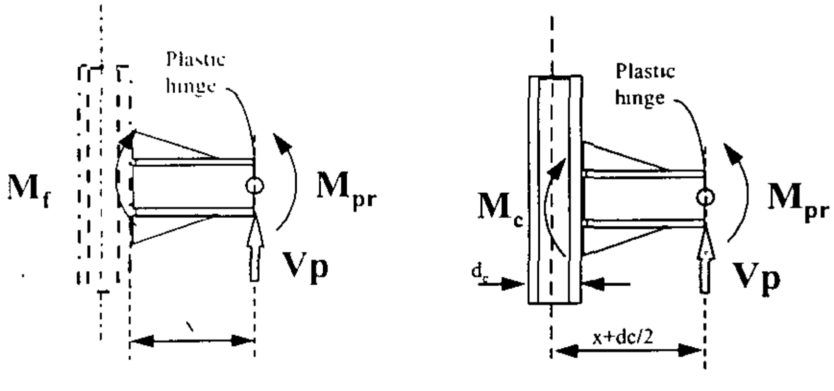
Note: if $2M_{pr}/L'$ is less then the gravity shear in the free body (in this case $P/2 + wL'/2$), then the plastic hinge location will shift and L' must be adjusted, accordingly

taking the sum of moments about "A" = 0
$Vp = \{Mpr + Mpr + P\ L'/2 + wL'^{2}/2\}/L'$

~~Figure 6-14~~ Figure 6.6.3.4-1 - Sample Calculation of Shear at Plastic Hinge

6.6.6.3.5 Determine Strength Demands on Connection

In order to complete the design of the connection, including sizing the various plates and joining welds which make up the connection, it is necessary to determine the shear and flexural strength demands at each critical section. These demands may be calculated by taking a free body of that portion of the connection assembly located between the critical section and the plastic hinge. ~~Figure 6-15~~ Figure 6.6.3.5-1 demonstrates this procedure for two critical sections, for the beam shown in ~~Figure 6-14~~Figure 6.6.3.4-1.



$$M_f = M_{pr} + V_p x$$

$$M_c = M_{pr} + V_p(x + d_c/2)$$

Critical Section at Column Face — Critical Section at Column Centerline

~~Figure 6-15~~ Figure 6.6.3.5-1 - Calculation of Demands at Critical Sections

Commentary: Each unique connection configuration may have different critical sections. The vertical plane that passes through the joint between the beam flanges and column (if such joining occurs) will typically define at least one such critical section, used for designing the joint of the beam flanges to the column, as well as evaluating shear demands on the column panel zone. A second critical section occurs at the center line of the column. Moments calculated at this point are used to check weak beam - strong column conditions. Other critical sections should be selected as appropriate to the connection configuration.

6.6.6.3.6 Check for Strong Column - Weak Beam Condition

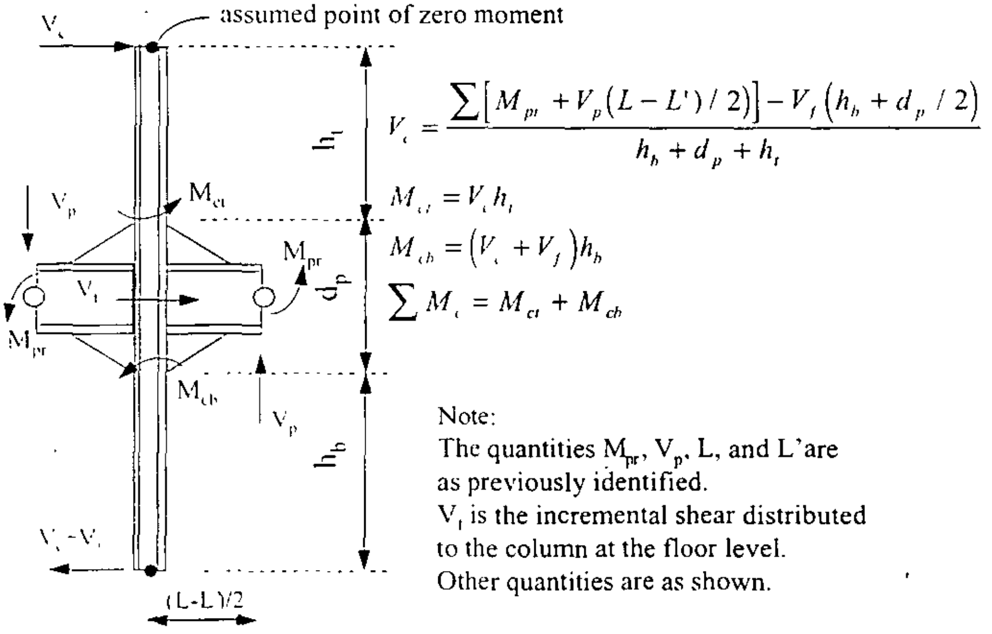
Buildings which formsidesway mechanisms through the formation of plastic hinges in the beams can dissipate more energy than buildings that develop mechanisms consisting primarily of plastic hinges in the columns. Therefore, if an existing building's original design was such that hinging would occur in the beams rather than the columns, care should be taken not to alter this behavior with the addition of connection reinforcement. To determine if the desired strong column - weak beam condition exists, the connection assembly should be checked to determine if the following equation is satisfied:

$$\sum Z_c(F_{yc} - f_a)/\sum M_c > 1.0$$ (6.6.6.3.6-1~~2~~)

where. Z_c is the plastic modulus of the column section above and below the connection
F_{yc} is the minimum specified yield stress for the column above and below
f_a is the axial load in the column above and below
ΣM_c is the ~~moment calculated at the center of the column in accordance with Section 6.6.6.3.5~~ sum of the column moments at the top and bottom of the panel zone, respectively, resulting from the development of the probable beam plastic moments, M_{pr}, within each beam in the connection.

Commentary: Equation 6.6.6.3.6-1~~2~~ is based on the building code provisions for strong column - weak beam design. The building code provisions for evaluating strong column - weak beam conditions presume that the flexural stiffness of the columns above and below the beam are approximately equal, that the beams will yield at the face of the column, and that the depth of the columns and beams are small relative to their respective span lengths. This permits the code to use a relatively simple equation to evaluate strong column - weak beam conditions in which the sum of the flexural capacities of columns at a connection are compared to the sums of the flexural capacities in the beams. The first publication of the Interim Guidelines took this same approach, except that the definition of ΣM_c was modified to explicitly recognize that because flexural hinging of the beams would occur at a location removed from the face of the column, the moments delivered by the beams to the connection would be larger than the plastic moment strength of the beam. In this equation, ΣM_c was taken as the sum of the moments at the *center of the column, calculated in accordance with the procedures of Sect. 6.6.3.5.*

This simplified approach is not always appropriate. -If non-symmetrical connection configurations are used, such as a haunch on only the bottom side of the beam, this can result in an uneven distribution of stiffness between the two column segments, and premature yielding of the column, either above, or below, the beam-column connection. Also, it was determined that for connection configurations in which the panel zone depth represents a significant fraction of the total column height, such as can occur in some haunched and side-plated connections, the definition of ΣM_c contained in the initial printing of the Guidelines could lead to excessive conservatism in determining whether or not a strong column - weak beam condition exists in a structure. Consequently, Interim Guidelines Advisory No. 1 adopted the current definition of ΣM_c for use in this evaluation. This definition requires that the moments in the column, at the top and bottom of the panel zone be determined for the condition when a plastic hinge has formed at all beams in the connection. Figure 6.6.6.3.6-1 illustrates a method for determining this quantity. ~~In such cases,~~ When evaluation indicates that a strong column - weak beam condition does not exist, a plastic analysis should be considered to determine if an undesirable story mechanism is likely to form in the building.



$$V_c = \frac{\sum\left[M_{pr} + V_p(L - L')/2)\right] - V_f\left(h_b + d_p/2\right)}{h_b + d_p + h_t}$$

$$M_{ct} = V_c h_t$$

$$M_{cb} = \left(V_c + V_f\right)h_b$$

$$\sum M_c = M_{ct} + M_{cb}$$

Note:
The quantities M_{pr}, V_p, L, and L'are as previously identified.
V_f is the incremental shear distributed to the column at the floor level.
Other quantities are as shown.

Figure 6.6.6.3.6-1 Calculation of Column Moment for Strong Column Evaluation

6.6.6.3.7 Check Column Panel Zone

The adequacy of the shear strength of the column panel zone should be checked. For this purpose, the term $0.8\Sigma M_f$ should be substituted for the term $0.8\Sigma M_s$ in *UBC-94* Section 2211.7.2.1 {$0.9\Sigma\phi_b M_p$ in *NEHRP-91* Section 10.10.3.1} ~~repeated below for convenience of reference~~. M_f is the calculated moment at the face of the column, when the beam mechanism forms, calculated as indicated in Section 6.6.6.3.5, above. In addition, it is recommended not to the alternative design criteria indicated in *UBC-94* Section 2211.7.2.1 (*NEHRP-91* Sect. 8.10.3.1), permitting panel zone shear strength to be proportioned for the shear induced by bending moments from gravity loads plus 1.85 times the prescribed seismic forces. For convenience of reference, UBC-94 Section 2211 7.2.1 is reproduced below, edited, to indicate the recommended application:

> **2211.7.2.1 Strength** (edited). The panel zone of the joint shall be capable of resisting the shear ~~induced by beam bending moments due to gravity loads plus 1.85 times the prescribed seismic forces, but the shear strength need not exceed that~~ required to develop ~~$0.8\Sigma M_s$~~ - $0.8\Sigma M_f$ of the girders framing into the column flanges at the joint. The joint panel zone shear strength may be obtained from the following formula:

$$V = 0.55F_y d_c t\left[1 + \frac{3b_c t_{cf}^{\,2}}{d_b d_c t}\right] \qquad (11\text{-}1)$$

where. b_c = width of column flange
d_b = the depth of the beam (including haunches or cover plates)
d_c = the depth of the column
t = the total thickness of the panel zone including doubler plates
t_{cf} = the thickness of the column flange

Commentary: The effect of panel zone shear yielding on connection behavior is not well understood. In the past, panel zone shear yielding has been viewed as a benign mechanism that permits overall frame ductility demands to be accommodated while minimizing the extent of inelastic behavior required of the beam and beam flange to column flange joint. The criteria permitting panel zone shear strength to be proportioned for the shears resulting from moments due to gravity loads plus 1.85 times the design seismic forces was adopted by the code specifically to encourage designs with weak panel zones. However, during recent testing of large scale connection assemblies with weak panel zones, it has been noted that in order to accommodate the large shear deformations that occur in the panel zone, extreme "kinking" deformations were induced into the column flanges at the beam flange to column flange welded joint. While this did not lead to premature joint failure in all cases, it is believed to have contributed to such premature failures in at least some of the specimens. The recommendations of this section are intended to result in stronger panel zones than previously *permitted by the code, thereby avoiding potential failures due to this kinking action on the column flanges.*

6.6.7 Modification Details

There are no modifications to the Guidelines or Commentary of Section 6.6.7 at this time.

6.6.7.1 Haunch at Bottom Flange

Figure ~~6-16~~6.6.7.1-1 illustrates the basic configuration for a connection modification consisting of the addition of a welded haunch at the bottom beam flange. Several tests of such a modification were conducted by Uang under the SAC phase I project (Uang, 1995). Following that work, additional research on the feasibility of improving connection performance with welded haunches was conducted under a project that was jointly sponsored by NIST and AISC (*NIST, 1998*). As indicated in the report of that work, the haunched modification improves connection performance by altering the basic behavior of the connection. In essence, the haunch creates a prop type support, beneath the beam bottom flange. This both reduces the effective flexural stresses in the beam at the face of the support, and also greatly reduces the shear that must be transmitted to the column through the beam. Laboratory tests indicate this modification can be effective when the existing low-toughness welds between the beam bottom flange and column are left in place, however, more reliable performance is obtained when the top welds are modified. A complete procedure for the design of this modification may be found in *NIST, 1998*~~. two alternative configurations of this detail that have been tested (Uang 1995). The basic concept is to reinforce the connection with the provision of a triangular haunch at the bottom flange. The intended behavior of both configurations is to shift the plastic hinge from the face of the column and to reduce the demand on the CJP weld by increasing the effective depth of the section. In one test, shown on the left of Figure 6-16, the joint between the girder bottom flange and column was cut free, to simulate a condition which might occur if the bottom joint had been damaged, but not repaired. In a second tested configuration, the bottom flange joint was repaired and the top flange was replaced with a locally thickened plate, similar to the detail shown in Figure 6-9~~.

Design Issues: This approach developed acceptable levels of plastic rotation. Acceptable levels of connection strength were also maintained during large inelastic deformations of the plastic hinge. This approach does not require that the top flange be modified, or slab disturbed, unless other conditions require repair of the top flange~~, as in the detail on the left of Figure 6-16~~. The bottom flange is generally far more accessible than the top flange because a slab does not have to be removed. In addition, the haunch can be installed at perimeter frames without removal of the exterior building cladding. There did not appear to be any appreciable degradation in performance when the bottom beam flange was not re-welded to the face of the column. Eliminating this additional welding should help reduce the cost of the repair.

Performance is dependent on properly executed complete joint penetration welds at the column face and at the attachment of the haunch to the girder bottom flange. The joint can be subject to through-thickness flaws in the column flange; however, this connection may not be as sensitive *to this potential problem because of the significant increase in the effective depth of the beam section which can be achieved. Welding of the bottom haunch requires overhead welding. The skewed groove welds of the haunch flanges to the girder and column flanges may be difficult to execute.*

Experimental Results: *~~This approach developed excellent levels of plastic rotation. In Specimen 1, the bottom flange CJP weld was damaged in a prior test but was not repaired; only the bottom haunch was added. During the test of specimen 1, a slowly growing crack developed at the underside of the top flange-web intersection, perhaps exacerbated by significant local buckling on the top flange. Some of the buckling may be attributed to lateral torsional buckling that occurred because the bottom flange was not restrained by a CJP weld. A significant portion of the flexural strength was lost during the cycles of large plastic rotation. In the second specimen, the bottom girder flange weld was intact during the haunch testing, and its performance was significantly improved compared with the first specimen. The test was stopped when significant local buckling led to a slowly growing crack at the beam flange and web intersection. At this time, it appears that repairing damaged bottom flange welds in this configuration can produce better performance. Acceptable levels of flexural strength were maintained during large inelastic deformations of the plastic hinge for both specimens.~~ As reported in NIST, 1998, a total of 9 beam-column connection tests incorporating bottom haunch modifications of pre-Northridge connections have been tested in the laboratory, including two dynamic tests. Most of the connection assemblies tested resisted in excess of 0.02 radians of imposed plastic rotation. However, for those specimens in which the existing low-toughness weld was left in place at the beam top flange, without modification, connection behavior was generally limited by fractures generating at these welds at relatively low plastic rotations. It may be expected that enhanced performance can be obtained by replacing or reinforcing these welds as part of the modification.*

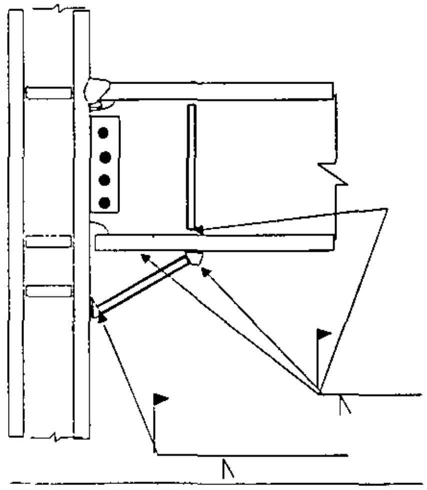


Figure 6-166.6.7.1-1 - **Bottom Haunch Connection Modification**

Post-earthquake Repair and Modification

Quantitative Results: *No. of specimens tested: ~~2~~9*
Girder Size: W30 x 99
Column Size: W14 x 176
Plastic Rotation achieved-
Specimen ~~1~~ UCSD-1R: 0.04 radian (w/o bottom flange weld)
Specimen ~~2~~ UCSD-3R: 0.05 radian (with bottom flange weld)
Specimen UCSD-4R: 0.014 radian (dynamic- limited by test setup)
Speciemn UCSD-5R: 0.015 radian (dynamic- limited by test setup)

Girder Size: W36x150
Column Size. W14x257
Plastic Rotation achieved -
Specimen UCB-RN2: 0.014 radian (no modification of top weld)
Specimen UTA-1R: 0.019 radian (partial modification of top weld)
Specimen UTA-1RB: 0.028 radian (modified top weld)

Girder Size: W36x150
Column Size: W14x455
Plastic Rotation achieved-
Specment UTA-NSF4: 0.015 radian (no modification of top weld)

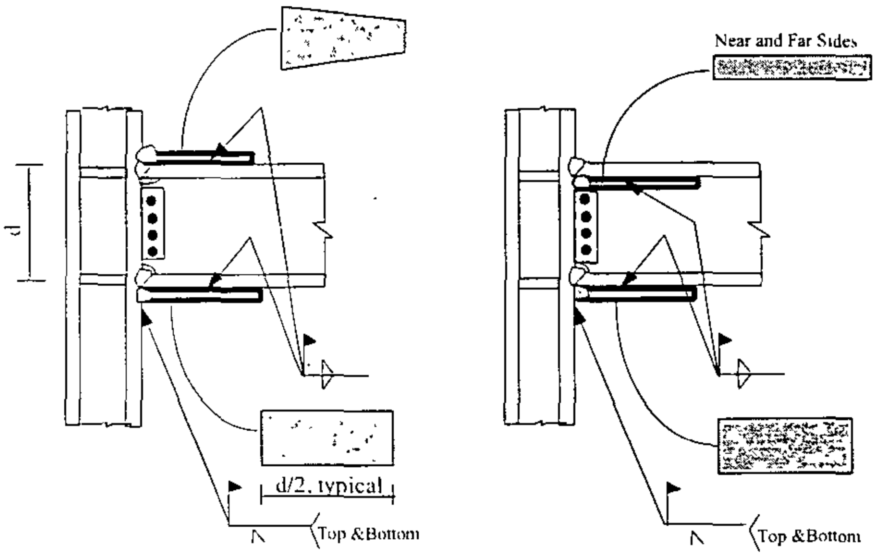
Girder Size: W18x86
Column Size: W24x279
Plastic Rotation achieved-
Specimen SFCCC-8: 0.035 radian (cover plated top flange)

6.6.7.2 Top and Bottom Haunch

There are no modifications to the Guidelines or Commentary of Section 6.6.7.2 at this time.

6.6.7.3 Cover Plate Sections

Figure 6.6.7.3-1 ~~Figure 6-18~~ illustrates the basic configurations of cover plate connections. The assumption behind the cover plate is that it reduces the applied stress demand on the weld at the column flange and shifts the plastic hinge away from the column face. Only the connection with cover plates on the top of the top flange has been tested. There are no quantitative results for cover plates on the bottom side of the top flange, such as might be used in repair. It is likely that thicker plates would be required where the plates are installed on the underside of the top flange. The implications of this deviation from the tested configuration should be considered.



~~Figure 6-18~~ Figure 6.6.7.3-1 - **Cover Plate Connection Modification**

Design Issues: Following the Northridge earthquake, the University of Texas at Austin conducted a program of research, under private funding, to develop a modified connection configuration for a specific project. Following a series of unsuccessful tests on various types of connections. ~~Approximately~~ eight connections similar to that shown in ~~Figure 6-18~~Figure 6.7.3-1 ~~have been~~ were tested (Engelhardt & Sabol - 1994), and ~~have~~ demonstrated the ability to achieve acceptable levels of plastic rotation provided that the beam flange to column flange welding was~~is~~ correctly executed and through-thickness problems in the column flange were~~are~~ avoided. Due to the significant publicity that followed these successful tests, as well as the economy of these connections relative to some other alternatives, cover plated connections quickly became the predominant configuration used in the design of new buildings. As a result, a number of qualification tests have now been performed on different variations of cover plated connections, covering a wide range of member sizes ranging from W16 to W36 beams, as part of the design process for individual building projects. The results of these tests have been somewhat mixed, with a significant number of failures reported. Although this connection type appears to be significantly more reliable than the typical pre-Northridge connection, it should be expected that some connections in buildings incorporating this detail may still be subjected to earthquake initiated fracture damage. Designers should consider using alternative connection types, unless highly redundant framing systems are employed.

The option with the top flange cover plate located on top of the flange can be used on perimeter frames where access to the outer side of the beam is restricted by existing building cladding. The option with the cover plate for the top flange located beneath the flange can be installed without requiring modification of the slab. In the figures shown, the bottom cover plate is rectangular, and sized slightly wider than the beam flange to allow downhand fillet welding of the joint between the two plates. Some configurations using triangular plates at the bottom flange, similar to the top flange have also been tested.

Designers using this detail are cautioned to be mindful of not making cover plates so thick that excessively large welds of the beam flange combination to column flange result. As the cover plates increase in size, the weld size must also increase. Larger welds invariably result in greater shrinkage stresses and increased potential for cracking prior to actual loading. In addition, larger welds will lead to larger heat affected zones in the column flange, a potentially brittle area

Performance is dependent on properly executed girder flange welds. The joint can be subject to through-thickness failures in the column flange. Access to the top of the top flange requires demolition of the existing slab. Access to the bottom of the top flange requires overhead welding and may be problematic for perimeter frames. Costs are greater than those associated with approaches that concentrate modifications on the bottom flange

Experimental Results: Six of eight connections tested by the University of Texas at Austin were able to achieve plastic rotations of at least 0.025 radians, or better. These tests were performed using heavy column sections which forced nearly all of the plastic deformation into the beam plastic hinge; very little column panel zone deformation occurred. Strength loss at the extreme levels of plastic rotation did not reduce the flexural capacity to less than the plastic moment capacity of the section based on minimum specified yield strength. One specimen achieved plastic rotations of 0.015 radians when a brittle fracture of the CJP weld (type W2 failure) occurred. This may partially be the result of a weld that was not executed in conformance with the specified welding procedure specification. The second unsuccessful test specimen achieved plastic rotations of 0.005 radian when a section of the column flange pulled out (type C2 failure). The successful tests were terminated either when twisting of the specimen threatened to damage the test setup or the maximum stroke of the loading ram was achieved. Since the completion of that testing, a number of additional tests have been performed. Data for 18 tests, including those performed by Engelhardt and referenced above, are in the public domain. At least 12 other tests have been performed on behalf of private parties, however, the data from these tests are not available. Some of those tests exhibited premature fractures.

Quantitative Results: *No. of specimens tested: 18*
Girder Size. W21 x 68 to W36 x 150
Column Size: W12 x 106 to W14 x 455~~, and 426~~
Plastic Rotation achieved-
~~6~~ *13 Specimens : >.025 radian to 0.05 radian*
~~1~~*3 Specimens:* $0.005 < \theta_p < 0.025$~~0.015 radian (W2 failure)~~
~~1~~*2 Specimens: 0.005 radian* ~~(C2 failure)~~

6.6.7.4 Upstanding Ribs

There are no modifications to the Guidelines or Commentary of Section 6.6.7.4 at this time.

6.6.7.5 Side-Plate Connections

There are no modifications to the Guidelines or Commentary of Section 6.6.7.5 at this time.

6.6.7.6 Bolted Brackets

Heavy bolted brackets, incorporating high strength bolts, may be added to existing welded connections to provide an alternative load path for transfer of stress between the beams and columns. To be compatible with existing welded connections, the brackets must have sufficient strength and rigidity to transfer beam stresses with negligible deformation. Pre-tensioning of the bolts or threaded rods attaching the brackets to the column flanges and use of welds or slip-critical connections between the brackets and beam flanges can help to minimize deformation under load. Reinforcement of the column flanges may be required to prevent local yielding and excessive deformation of these elements. Two alternative configurations, which may be used either to repair an existing damaged, welded connection or to reinforce an existing undamaged connection are illustrated in Figure 6.6.7.6-1. The developer of these connections offers the brackets in the form of proprietary steel castings. Several tests of these alternative connections have been performed on specimens with beams ranging in size from W16 to W36 sections and with large plastic rotations successfully achieved. Under a project jointly funded by NIST and AISC, the use of a single bracket at the bottom flange of the beam was investigated. It was determined that significant improvement in connection behavior could be obtained by placing a bracket at the bottom beam flange and by replacing existing low-toughness welds at the top flange with tougher material. *NIST, 1998* provides a recommended design procedure for such connection modifications.

Design Issues: The concept of bolted bracket connections is similar to that of the riveted "wind connections" commonly installed in steel frame buildings in the early twentieth century. The primary difference is that the riveted wind connections were typically limited in strength either by flexural yielding of outstanding flanges of the brackets, or shear and tension on the rivets, rather than by flexural hinging of the connected framing. Since the old-style wind connections could not typically develop the flexural strength of the girders and also could be quite flexible, they would be classified either as partial strength or partially restrained connections. Following the Northridge earthquake, the concept of designing such connections with high strength bolts and heavy plates, to behave as fully restrained connections, was developed and tested by a private party who has applied for patents on the concept of using steel castings for this purpose.

Bolted bracket connections can be installed in an existing building without field welding. Since this reduces the risk of construction-induced fire, brackets may be installed with somewhat less demolition of existing architectural features than with welded connections. In addition, the quality assurance issues related to field welding are eliminated. However, the fabrication of the brackets themselves does require quality assurance. Quality assurance is also required for operations related to the drilling of bolt holes for installation of bolts, surface preparation of faying surfaces and for installation and tensioning of the bolts themselves.

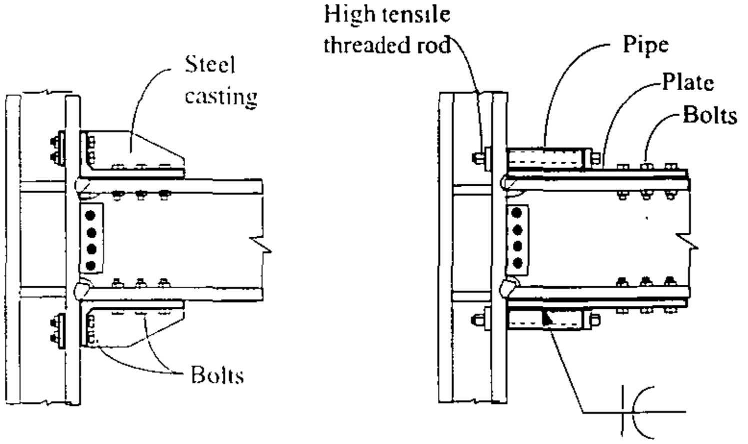


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Figure 6.6.7.6-1 Bolted Bracket Modification

Bolted brackets can be used to repair damaged connections. If damage is limited to the beam flange to column flange welds, the damaged welds should be dressed out by grinding. Any existing fractures in base metal should be repaired as indicated in Section 6.3, in order to restore the strength of the damaged members and also to prevent growth of the fractures under applied stress. Since repairs to base metal typically require cutting and welding, this reduces somewhat the advantages cited above, with regard to avoidance of field welding.

Experimental Results. A series of tests on several different configurations of proprietary heavy bolted bracket connections have been performed at Lehigh University (Ksai & Bleiman, 1996) to qualify these connections for use in repair and modification applications. To test repair applications, brackets were placed only on the bottom beam flange to simulate installations on a connection where the bottom flange weld in the original connection had failed. In these specimens, bottom flange welds were not installed, to approximate the condition of a fully fractured weld. The top flange welds of these specimens were made with electrodes rated for notch toughness, to preclude premature failure of the specimens at the top flange. For specimens in which brackets were placed at both the top and bottom beam flanges, both welds were omitted. Acceptable plastic rotations were achieved for each of the specimens tested. No testing has yet been performed to determine the effectiveness of bolted brackets when applied to an existing undamaged connection with full penetration beam flange to column flange welds with low toughness or significant defects or discontinuities.

Quantitative Results. No. of specimens tested: 8
Girder Size: W16x40 and W36x150
Column Size: W12x65 and W14x425
Plastic Rotation achieved - 0.05 radians - 0.07 radians

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XXVI CURSO INTERNACIONAL DE INGENIERIA SÍSMICA

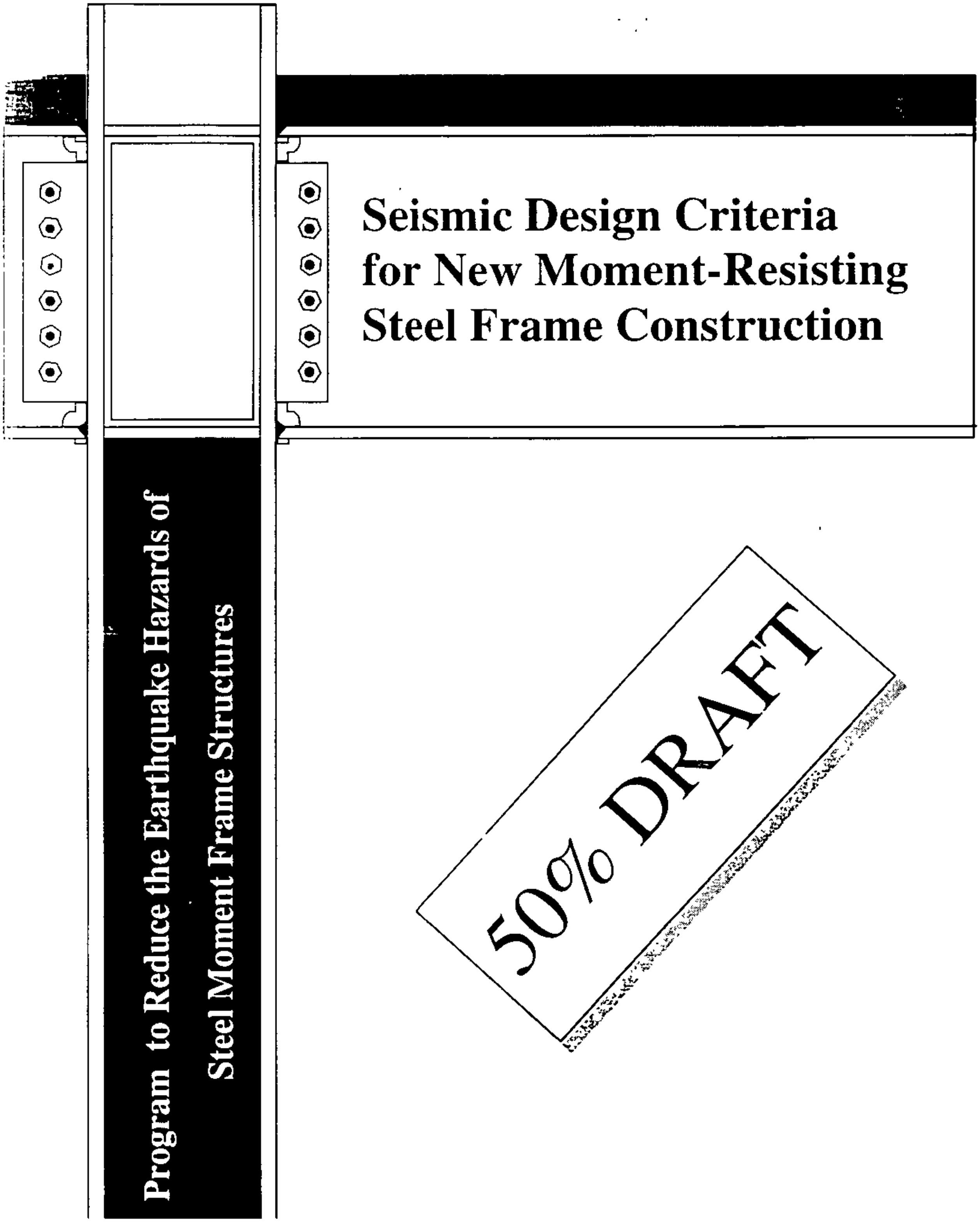
MOUDLO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

SEISMIC DESIGN CRITERIA FOR NEW MOMENT-RESISTING STEEL FRAME CONSTRUCTION

EXPOSITOR: DR. DAVID DE LEON ESCOBEDO
PALACIO DE MINERIA
SEPTIEMBRE DEL 2000

FEDERAL EMERGENCY MANAGEMENT AGENCY FEMA XXX/January, 1999



Seismic Design Criteria for New Moment-Resisting Steel Frame Construction

Program to Reduce the Earthquake Hazards of Steel Moment Frame Structures

50% DRAFT

Observation of damage sustained by buildings in the Northridge Earthquake indicates that contrary to the intended behavior, in many cases brittle fractures initiated within the connections at very low levels of plastic demand, and in some cases, while the structures remained elastic. Typically, but not always, fractures initiated at, or near, the complete joint penetration (CJP) weld between the beam bottom flange and column flange (Figure 1-1). Once initiated, these fractures progressed along a number of different paths, depending on the individual joint conditions.

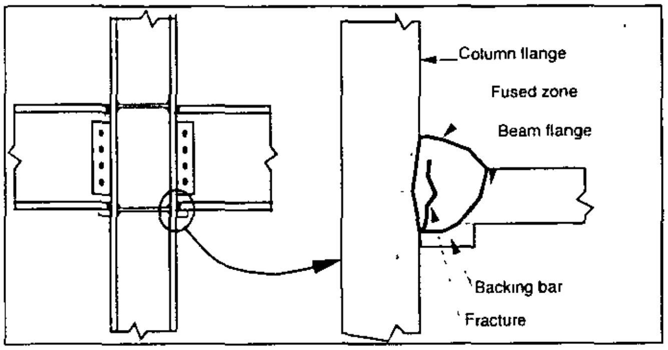
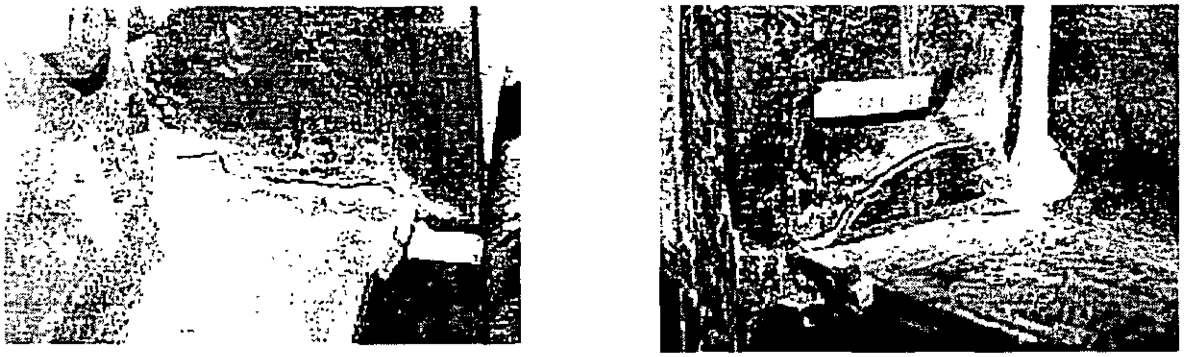


Figure 1-1 - Common Zone of Fracture Initiation in Beam -Column Connection

In some cases, the fractures progressed completely through the thickness of the weld, and if fire protective finishes were removed, the fractures were evident as a crack through exposed faces of the weld, or the metal just behind the weld (Figure 1-2a). Other fracture patterns also developed. In some cases, the fracture developed into a crack of the column flange material behind the CJP weld (Figure 1-2b). In these cases, a portion of the column flange remained bonded to the beam flange, but pulled free from the remainder of the column. This fracture pattern has sometimes been termed a "divot" or "nugget" failure.

A number of fractures progressed completely through the column flange, along a near horizontal plane that aligns approximately with the beam lower flange (Figure 1-3a). In some cases, these fractures extended into the column web and progressed across the panel zone Figure (1-3b). Investigators have reported some instances where columns fractured entirely across the section.



a. Fracture at Fused Zone

b. Column Flange "Divot" Fracture

Figure 1-2 - Fractures of Beam to Column Joints

a. Fractures through Column Flange

b. Fracture Progresses into Column Web

Figure 1-3 - Column Fractures

Once such fractures have occurred, the beam - column connection has experienced a significant loss of flexural rigidity and strength to resist loads that tend to open the crack. Residual flexural strength and rigidity must be developed through a couple consisting of forces transmitted through the remaining top flange connection and the web bolts. However, in providing this residual strength and stiffness, the bolted web connections can themselves be subject to failures, consisting of fracturing of the welds of the shear plate to the column, fracturing of supplemental welds to the beam web or fracturing through the weak section of shear plate aligning with the bolt holes (Figure 1-4).

Figure 1-4 - Vertical Fracture through Beam Shear Plate Connection

Despite the obvious local strength impairment resulting from these fractures, many damaged buildings did not display overt signs of structural damage, such as permanent drifts, or damage to architectural elements, making reliable post-earthquake damage evaluations difficult. Until news of the discovery of connection fractures in some buildings began to spread through the engineering community, it was relatively common for engineers to perform cursory post-earthquake evaluations

3.4.1 Welded Unreinforced Flange (WURF)

This section provides guidelines for design of unreinforced, welded flange connections. These connections utilize complete joint penetration (CJP) groove welds, with improvements over those used prior to the Northridge earthquake, to join beam or girder flanges directly to column flanges. In this type of connection, no reinforcement other than weld metal, is used to join the flanges. Web joints for these connections are CJP welded. This type of connection should be used only for OMF applications, when such systems are permitted by the building code, or when factored drift demands predicted by structural analysis, conducted in accordance with Chapter 4, can be shown to be lower than the product of $\phi_I\theta_I$, where θ_I and ϕ_I, respectively are the values indicated in Tables 3-2 and 3-3 for connection drift capacity and capacity reduction factor for the appropriate performance level. Figure 3-4 provides a typical detail for this connection type. These connections should be designed in accordance with the guidelines of this section.

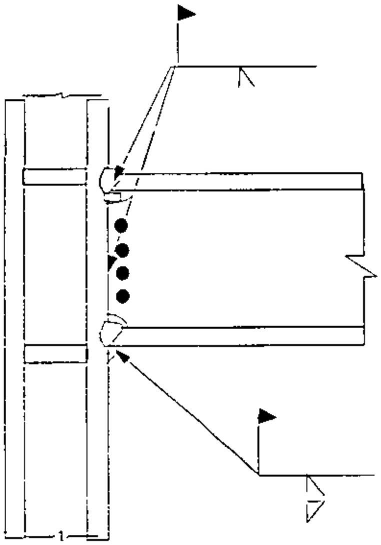


Figure 3-4 - Typical Detail - WUF Connection

Table 3-3 - Pre-qualification Data WUF Connections

Applicable systems	OMF
Pre-qualified Drift Angle Capacity	0.02 radian - collapse prevention 0.015 radian - incipient damage
Capacity Reduction Factor o	0 6 - collapse prevention 0 9 - incipient damage
Hinge location distance s_h	d/2
Maximum beam size	W36 x 150
Beam Material	A36, A572, Gr 50, A913 Grade 50, A992
Cover Plate Material	Minimum specified yield strength equal or larger than that specified

Michigan, none of the specimens achieved drift angles in excess of 0.025 radians.

3.4.2 Welded Cover Plated Flanges (WCPF)

This section provides guidelines for design of welded cover plated flange connections. These connections utilize complete penetration groove welds, with improvements over those used prior to the Northridge earthquake, to join beam or girder flanges and top and bottom flange cover plates directly to column flanges. Web joints for these connections are welded. This type of connection should be used only when inelastic rotation demands can be shown to be lower than value indicated in Table 3-4. Figure 3-5 provides a typical detail for this connection type. These connections should be designed in accordance with the guidelines of this section.

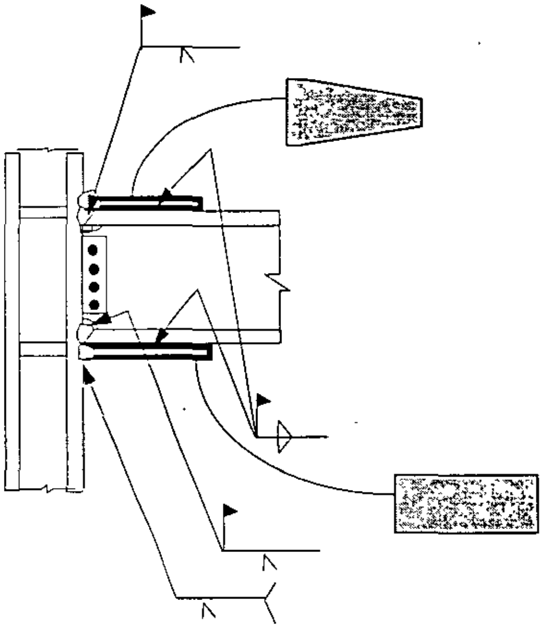


Figure 3-5 Typical Detail WCPF Connection

Table 3-4 Pre-qualification Data for WCPF Connections

Applicable systems	OMF, IMF, SMF
Pre-qualified Drift Angle Capacity	0.04 radian - collapse prevention 0.015 radian - incipient damage
Capacity Reduction Factor o	0.75 - collapse prevention 0.9 - incipient damage
Hinge location distance s_h	$d_c/2 + l_p + d_b/4$
Maximum beam size	W36 x
Beam Material	A36, A572, Gr. 50, A913 Grade 50 or 65, A992

Maximum column size	unlimited
Column Steel Grades	ASTM A-572, Gr. 50. ASTM A-913, Grade 50 or 65, ASTM A-992
Panel zone to Beam strength ratio	*to be determined*
Column to Beam strength ratio	*to be determined*

3.4.2.1 Procedure for Sizing Cover Plates

Cover plates for this type of connection should have an area of about ¾ of that of the beam flange.

3 4.2.2 Procedure for Sizing Shear Tabs

Shear tabs for this type of connection should be sized with minimum thickness, bolts and welds to the columns, as required to resist erection loads. The shear tab serves as a backing for the CJP web weld and therefore, it should be continuously fillet or groove welded on the side away from the CJP weld.

3 4 2.3 Procedure for Weld Sizing / Weld Configuration

<<<<< to be developed >>>>>>

3 4.2.4 Continuity Plate Sizing

Continuity Plates should be sized using the guidelines of Section 3.3.3.1, considering the beam flange thickness to be equal to the thickness of the combined flange and cover plate.

3.4.3 Welded Flange Plates (WFP)

3.4.4 Reduced Beam Section (RBS, or Dog Bone)

This section provides guidelines for design of type FR reduced beam section connections. These connections utilize circular radius cuts in both top and bottom flanges to reduce the flange area over a length of the beam near the ends of the beam span Welds are complete penetration groove welds, with improvements over those used prior to the Northridge earthquake, to join beam or girder flanges directly to column flanges. In this type of connection, no reinforcement other than weld metal, is used to join the flanges. Web joints for these connections are welded. This type of connection should be used only when drift angle demands can be shown to be lower than the value indicated in Table 3-6. Figure 3-7 provides a typical detail for this connection type.

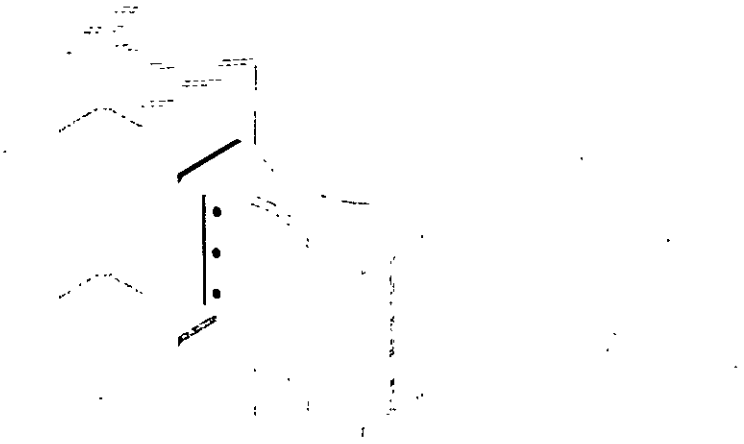


Figure 3-6 Typical RBS Connection (Ref Englehardt)

Table 3-5 Pre-qualification Data for RBS Connections

Applicable systems	OMF, IMF, SMF
Pre-qualified Drift Angle Capacity	0.04 radian - collapse prevention 0.015 radian - incipient damage
Capacity Reduction Factor φ	0.85 - collapse prevention 0.9 - incipient damage
Hinge location distance s_h	Center of Col. to Center of Reduced Sect.
Maximum beam size	W36 x 194 (Largest Tested)
Beam Steel Grades	ASTM A-572, Gr. 50, ASTM A-992
Maximum column size	unlimited
Column Steel Grades	ASTM A-572, Gr. 50, ASTM A-913, Grade 50 or 65, ASTM A-992
Panel zone to Beam strength ratio	*to be determined*
Column to Beam strength ratio	*to be determined*

3 4 4 1 Procedure for Sizing Section Reduction

Figure 3-7 shows the geometry of a radius cut RBS, and Fig. 3-8 shows the entire moment frame beam. The designer should select the dimensions *a* and *b* according to the following guidelines:

$$a \cong (0.5 \text{ to } 0.75)\, b_f \tag{3-4}$$

$$b \cong (0.65 \text{ to } 0.85)\, d \tag{3-5}$$

where b_f and d are the beam flange width and depth respectively.

The remaining dimension that must be chosen when sizing the RBS is c, the depth of the cut. The value of c will control the maximum moment developed within the RBS, and therefore will control the maximum moment and shears generated at the face of the column. The final dimensions should be chosen so that the maximum moment at the face of the column is in the range of about 85 to 100 percent of the beam's expected plastic moment. The value of c should be chosen to be less than or equal to $0.25b_f$.

The radius of the cut R can be related to dimensions b and c based on the geometry of a circular arc, using the equation in Fig. 3.8. The amount of flange material which is removed at the minimum section of the RBS is sometimes referred to the *percent flange removal* which is computed as $(2c/b_f) \times 100$, where b_f is the unreduced flange width of the beam.

Once dimensions have been selected based on the above guidelines, calculations using standard methods of strength of materials and the general guidelines given in sections 3.2 and 3.3 above can be used to verify that the required condition for maximum moment at the column face is met.

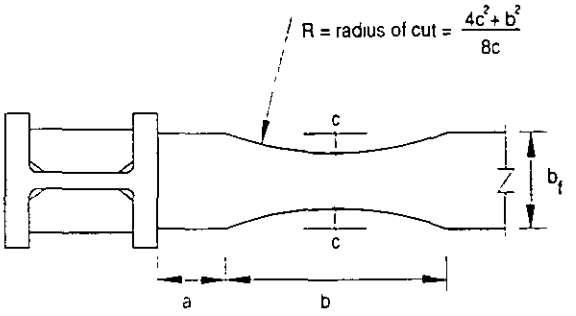


Fig. 3.7 Geometry of Radius Cut RBS (Ref. Englehardt)

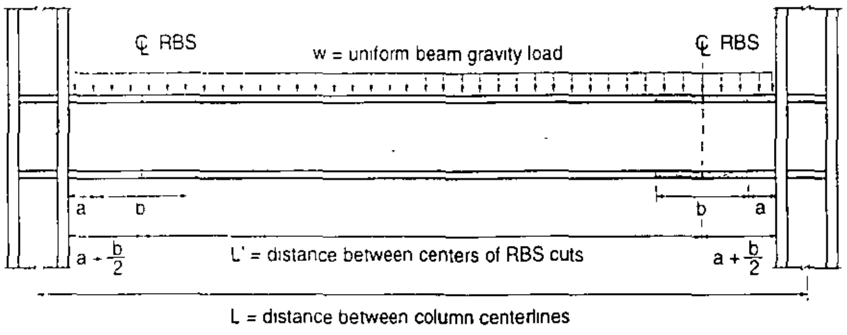


Fig. 3.8 Typical Moment Frame Beam with RBS Connections (Ref. Englehardt)

> *"Virtually all moment connections that dissipate energy by yielding of the beam are subject to varying degrees of beam instability at large levels of inelastic rotation. This is true both for reinforced connections (cover plates, ribs, haunches, etc.) and for RBS connections. This instability generally involves a combination of flange buckling, web buckling and lateral torsional buckling and typically results in a deterioration in the flexural strength of the beam with increasing inelastic rotations. In the experience of the writer, the degree of instability and associated strength deterioration for RBS connections tested in the laboratory have been no more severe, and perhaps somewhat less severe than for many types of reinforced connections. This is demonstrated by the connection test results shown in Fig. 8.*

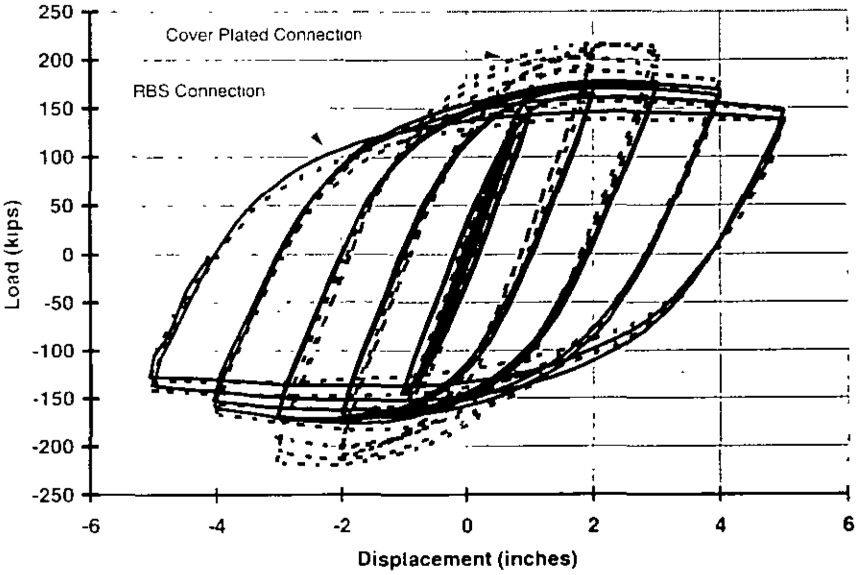


Fig.3-8 - Comparison of Test Results for Cover Plated and RBS Connections

This figure shows a plot of beam tip load versus beam tip displacement for two different test specimens (Refs. 2 and 15). These two specimens were virtually identical, except for the connection detail. Both specimens were constructed with the same member sizes (W36x150 beam and W14x426 column) and heats of steel, and tested in the same test setup with identical member lengths, identical member end support conditions, and identical lateral bracing. Both specimens were subject to the same loading history. The only difference was that one specimen was constructed with a cover plated connection and the other with an RBS connection. Both specimens were provided with a single beam lateral support near the point

3.5 Pre-qualified Bolted FR Connections

This section provides guidelines for four specific types of bolted, FR, MRSF connections suitable for different member sizes and with varying inelastic rotation capabilities, as indicated in Table 3-6. Depending on the rotation capacity required for the moment frame type being used and the member sizes needed, the designer may select a suitable connection from the table.

Table 3-6 - Pre-qualified Bolted FR Connections

Connection Type	Criteria Section	Frame Type	Incipient Damage		Collapse Prevention	
			Drift Angle	Reliability Factor Φ_I	Drift Angle	Reliability Factor Φ_c
BEP d ≤24"	3.5.1	OMF	0.02	0.9	.06	0.6
BEP d ≤18"	3.5 1	IMF	0.03	0.9	.07	0.6
WFPBB d ≤24"	3.5 2	OMF	0.02	0.9	.06	0.75
WFPBB d ≤18"	3.5 2	IMF	0.03	0 9	.06	0.75
BB	3.5.3	Proprietary connection[1]				

1- Specific qualification data for proprietary connections is not provided by these Guidelines. Refer to the licenser for specific information on the applicability of these connection types to various framing systems.

3.5.1 Bolted End Plate (BEP)

<<<<< to be developed >>>>>>

3.5.2 Welded Flange Plates with Bolted Beam (WFPBB)

This section provides guidelines for design of connections utilizing plates welded to column flanges and bolted to beam flanges. The flange plates are welded to the column flange using CJP welds following the recommendations given in sections 3.3.2.1 through 3.3.2.5. The flange plates are bolted to beam flanges following the recommendations of sections 3.3.3.1 and 3.3.3.2. A detail of this type of connection is shown in figure 3-11. The figure also shows various dimensions and nomenclature which is used in the design procedure which follows. Table 3-7 presents the range of pre-qualification for this connection type.

3.5.2 1 General Design Procedure

The behavior of this type connection can be controlled by a number of different modes including flexural yielding of the beam section, flexural yielding of the cover plates, net-section tensile failure of the beam flange or cover plates, shear failure of the

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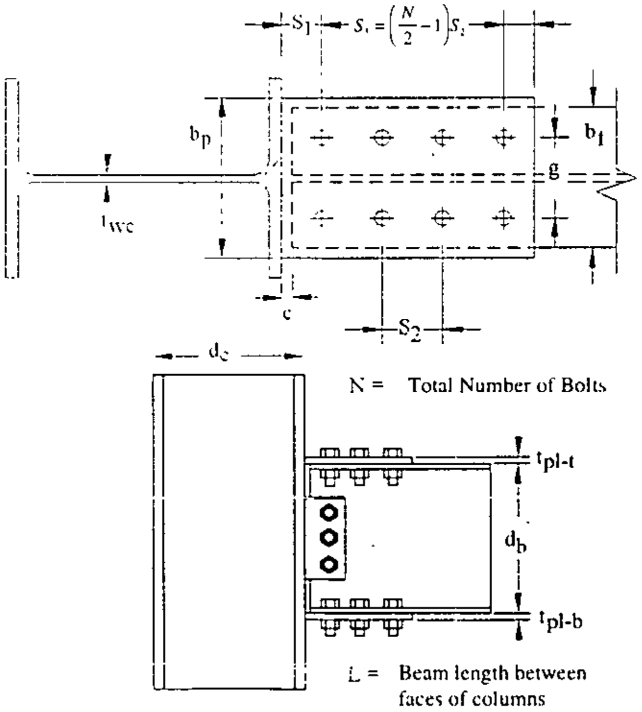
Table 3-7 Pre-qualification Data for WFPBB Connections

Applicable systems	Mode 1: OMF, IMF, SMF Mode 2: OMF, IMF
Pre-qualified Drift Angle Capacity	Mode 1: 0.04 radian - collapse prevention $\phi = 0.75$ 0.015 radian - incipient damage $\phi = 0.9$ Mode 2: 0.03 radian - collapse prevention $\phi = 0.75$ 0.015 radian - incipient damage $\phi = 0.9$
Hinge location distance s_h	Mode 1: - $S_1+S_3+d_b/2$ Mode 2: - $S_1/2$
Maximum beam size	W24 x 94
Beam Material	A36, A572, Gr. 50, A913 Grade 50 or 65, A992
Plate Material	A36, A572, Gr. 50, A992
Maximum column size	unlimited
Column Material	A36, A572, Gr. 50, A913 Grade 50 or 65, A992
Panel zone to Beam strength ratio	*to be developed*
Column to Beam strength ratio	*to be developed*



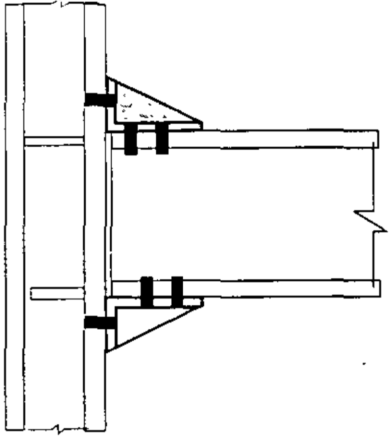


Figure 3-13 Bolted Bracket Connection

3.6 Pre-qualified PR Connections

This section provides guidelines for pre-qualified Partially Restrained (PR) MRSF connections suitable for different member sizes and with varying inelastic rotation capabilities, as indicated in Table 3-8. Depending on the rotation capacity required for the moment frame type being used and the member sizes needed, the designer may select a suitable connection from the types described in the following paragraphs. If a connection type other than one of the pre-qualified connections is to be used, it should be qualified by tests as described later in this section.

Table 3-8 - Pre-qualified Bolted PR Connections

Connection Type	Criteria Section	Frame Type	Incipient Damage		Collapse Prevention	
			Drift Angle - Incipient Damage	Reliability Factor Φ_I	Drift Angle - Incipient Damage	Reliability Factor Φ_c
DST d ≤30"	3.6.1	IMF			.08	0.75
DST d ≤24"	3.6.1	SMF			.10	0.75
STC d ≤30"	3.6.2					
STC d ≤24"	3.6.2					
SLC d ≤30"	3.6.3					
SLC d ≤24"	3.6.3	OMF			.10	0.75
SC d ≤24"	3.6.4	*			.10	0.85
SC d ≤18"	3.6.4	*			.12	0.9

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3.6.1 Double Split Tee Connections (DST)

This section provides guidelines for design of type PR connections employing bolted split tee connectors between the beam and column flanges. This type of connection should be used only when design parameters are within the limitations indicated in Table 3-9. Figure 3-14 provides a typical detail for this connection type. The design procedure of this section should apply.

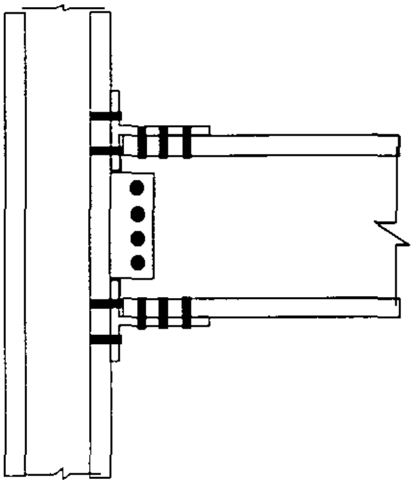

Figure 3-14- Typical Double Split Tee Connection

Table 3-9 - Pre-qualification Data for DST Connections

Applicable systems	OMF, IMF, SMF
Pre-qualified Inelastic Rotation Demand	0.10 radian - collapse prevention $\varphi = 0.75$ 0.03 radian - incipient damage $\phi = 0.9$
Hinge location distance s_h	$d_c/2$
Maximum beam size	W36 x 150
Beam Material	A36, A572, Gr. 50, A913 Grade 50 or 65, A992
Maximum column size	unlimited
Column Material	A36, A572, Gr. 50, A913 Grade 50 or 65, A992
Connection Stiffness	$10EI/L_b$
Connection Strength (fraction of Beam M_p)	30%

recommendations with regard to evaluating the performance of nonstructural components.

4.2.2.1 Structural Performance Levels

Two discrete structural performance levels are defined in these guidelines. Acceptance criteria, which relate to the permissible earthquake-induced forces and deformations for the various elements of MRSF structures, are tied directly to the structural performance levels. The performance levels are discrete damage states for which specific acceptance criteria are defined.

Structural Performance Levels are the Incipient Damage Level (S-1), and the Collapse Prevent Level (S-5). Table 4-2 relates these structural performance levels to the limiting damage states for common vertical elements of MRSF structures. Later sections of these Guidelines specify design parameters (inter-story drift ratios and component capacities) recommended as limiting values for calculated structural deformations and stresses for different structural components, in order to attain these structural performance levels for a known earthquake demand.

Table 4-2 - Structural Performance Levels

		Structural Performance Levels	
Elements	Type	Collapse Prevention S-5	Incipient Damage S-1
Girder		Extensive distortion. A few girders may experience fracture	Minor local yielding at a few places.
Column		Moderate distortion; some columns experience yielding. Some local buckling of flanges	No observable damage or distortion
Connection		Many fractures (X% of total ?) and/or extensive yielding	No observable fractures; minor yielding at some connections
Panel Zone		Extensive distortion	Minor distortion
Column Splice	Ductile Splices	Fractures at some locations	No yielding
Base Plate		Extensive yielding of anchor bolts and base plate	No observable damage or distortion
Drift	Inter-story	3% -6% depending on structural system	1% - 1-1/2% transient negligible permanent

4.2.2.1.1 Incipient Damage Performance Level (S-1)

Structural Performance Level S-1, Incipient Damage, means the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical and lateral force-resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness. The risk of life-threatening injury as a result of structural damage is very low, and although some minor structural repairs may be appropriate, these would generally not be required prior to re-occupancy.

4.2.2.1.2 Collapse Prevent Performance Level (S-5)

Structural Performance Level S-5, Collapse Prevention, is that performance level in which the structure is on the verge of experiencing partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral force-resisting system, large permanent lateral deformation of the structure, and to a more limited extent, degradation in the vertical load-carrying capacity. However, all significant components of the gravity load-resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling hazards from structural debris may exist. The structure may not be technically practical to repair and is not safe for re-occupancy, aftershock activity could credibly induce collapse.

> *Commentary: When a building is subjected to earthquake ground motion, a pattern of lateral deformations that varies with time is induced into the structure. At any given point in time, a particular state of lateral deformation will exist in the structure, and as some time within the period in which the structure is responding to the ground motion, a maximum pattern of deformation will occur. At relatively low levels of ground motion, the deformations induced within the building will be limited, and the resulting stresses which develop within the structural components will be within the elastic range of behavior. Within this elastic range, the structure will experience no damage. All structural components will retain their original strength, stiffness and appearance, and when the ground motion stops, the structure will return to its pre-earthquake condition.*
>
> *At more severe levels of ground motion, the lateral deformations induced into the structure will be larger. As these deformations increase, so will demands on the individual structural components. At different levels of deformation, corresponding to different levels of ground motion severity, individual components of the structure will be strained beyond their elastic range. As this occurs, the structure starts to experience damage in the form of buckling, yielding and fracturing of the various components. As components become damaged, they degrade in stiffness,*

and some elements will begin to lose their strength. In general, when a structure has responded to ground motion within this range of behavior, it will not return to its pre-earthquake condition when the ground motion stops. Some permanent deformation may remain within the structure and damage will be evident throughout. Depending on how far the structure has been deformed, and in what pattern, the structure may have lost a significant amount of its original stiffness and, possibly, strength.

Brittle elements are not able to sustain inelastic deformations and will fail suddenly; the consequences may range from local and repairable damage to collapse of the of the structural system. At higher levels of ground motion, the lateral deformations induced into the structure will strain a number of elements to a point at which the elements behave in a brittle manner, or as a result of the decreased overall stiffness, the structure loses stability. Eventually, partial or total collapse of the structure can occur. The structural performance levels relate the extent of a building's response to earthquake hazards to these various possible damage states.

At the Incipient Damage Level, damage is relatively limited. The structure retains a significant portion of its original stiffness and most if not all of its strength. At the Collapse Prevention level, the building has experienced extreme damage. If laterally deformed beyond this point, the structure can experience instability and collapse. FEMA-273 also includes consideration of a Life Safety level, intermediate between the damage states represented by Incipient Damage and Collapse Prevention. The Life Safety level is defined in FEMA-273 as occurring at 75% of the lateral displacement at which Collapse Prevention occurs. Given this circular definition of the Life Safety level, and the fact that the NEHRP Provisions have moved towards designing for Collapse Prevention, as opposed to Life Safety performance, this performance level has been omitted from these guidelines.

4.2.2.2 Nonstructural Performance Levels

Nonstructural Performance Levels are defined in these Guidelines for reference only. No specific guidelines are provided for attaining these performance levels. Refer to *FEMA-273* for more detailed information on the performance design of nonstructural components and systems.

4.2.2.2.1 Operational Performance Level (N-A)

Nonstructural Performance Level A, Operational, means the post-earthquake damage state of the building in which the nonstructural components are able to support the building's intended function. Under this level, most nonstructural systems required for normal use of the building including lighting, plumbing, HVAC, computer systems are functional although minor cleanup and repair of some items may be required. This performance level requires considerations beyond those that are normally within the sole province of the structural engineer. In addition to assuring that nonstructural components are properly mounted and braced within the structure, in order to achieve this performance, it is often necessary to provide emergency standby utilities. It may also be necessary to performance rigorous qualification testing of the ability of key electrical and mechanical equipment items to function during or after strong shaking.

4.2.2.2.2 Immediate Occupancy Level (N-B)

Nonstructural Performance Level B, Immediate Occupancy, means the post-earthquake damage state in which only limited nonstructural damage has occurred. Basic access and life safety systems, including doors, stairways, elevators, emergency lighting, fire alarms, and suppression systems, remain operable, provided that power is available. There could be minor window breakage and slight damage to some components. Presuming that the building is structurally safe, it is expected that occupants could safely remain in the building, although normal use may be impaired and some cleanup and inspection may be required. In general, components of mechanical and electrical systems in the building are structurally secured and should be able to function if necessary utility service is available. However, some components may experience misalignments or internal damage and be inoperable. Power, water, natural gas, communications lines, and other utilities required for normal building use may not be available. The risk of life-threatening injury due to nonstructural damage is very low.

4 2 2.2.3 Life Safety Level (N-C)

Nonstructural Performance Level C, Life Safety, is the post-earthquake damage state in which potentially significant and costly damage has occurred to nonstructural components but they have not become dislodged and fallen, threatening life safety either within or outside the building. Egress routes within the building are not extensively blocked, but may be impaired by lightweight debris. HVAC, plumbing, and fire suppression systems may have been damaged, resulting in local flooding as well as loss of function. While injuries may occur during the earthquake from the failure of nonstructural components, it is expected that overall, the risk of life-threatening injury is very low. Restoration of the nonstructural components may take extensive effort.

4 2 2.2.4 Hazards Reduced Level (N-D)

Nonstructural Performance Range D, Hazards Reduced, represents a post-earthquake damage state range in which extensive damage has occurred to nonstructural components, but large or heavy items that pose a falling hazard to a number of people such as parapets, cladding panels, heavy plaster ceilings, or storage racks are prevented from falling. While isolated serious injury could occur from falling debris, failures that could injure large numbers of persons, either inside or outside the structure, should be avoided. Exits,

Table 4-9 Confidence Levels for Various Values of γ_{con} for Different Analytical Approaches

Analysis Procedure	Linear Static Procedure					Linear Dynamic Procedure					Nonlinear Static Procedure					Nonlinear Dynamic Procedure				
Confidence Level	50	65	84	90	95	50	65	84	90	95	50	65	84	90	95	50	65	84	90	95
Geographic Region																				
California	.2	.5	1	1.2	1.5	.3	.6	1.1	1.3	1.5	.4	.7	1.1	1.3	1.5	.5	.8	1.2	1.3	1.5
Pacific N.W.	.6	.9	1.3	1.4	1.6	.5	.8	1.3	1.5	1.7	.4	.7	1.3	1.5	1.8	.3	.6	1.2	1.5	1.9
Intermountain	.6	.9	1.3	1.4	1.6	.5	.8	1.3	1.5	1.7	.4	.7	1.3	1.5	1.8	.3	.6	1.2	1.5	1.9
Central U.S.	.6	.9	1.4	1.6	1.7	.5	.9	1.5	1.7	2.0	.5	.8	1.5	1.8	2.2	.4	.8	1.6	1.9	2.4
Eastern U.S.	.6	.9	1.4	1.6	1.7	.5	.9	1.5	1.7	2.0	.5	.8	1.5	1.8	2.2	.4	.8	1.6	1.9	2.4

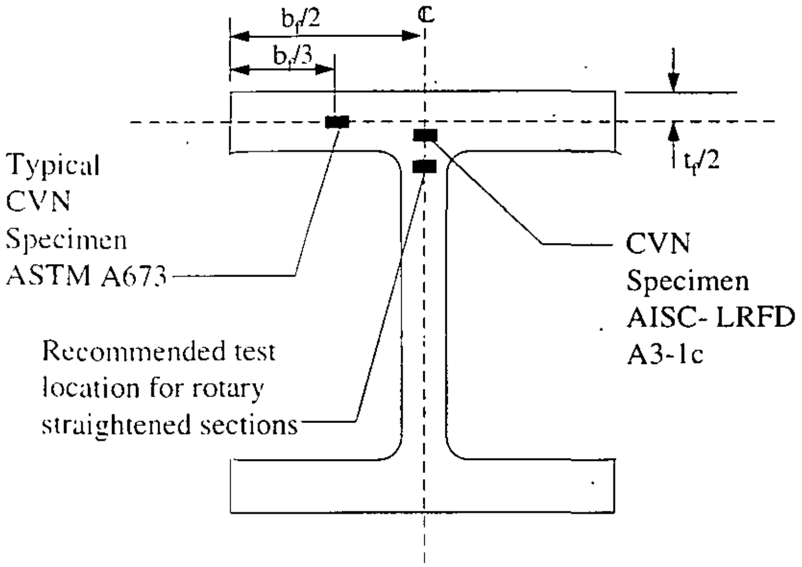

Figure 5-1 - Standard Locations for Charpy V-Notch Specimen Extraction, Longitudinal Only

Commentary: Specifying toughness properties in critical, unusual or non-redundant connections should be considered. As temperature decreases or strain rate increases, toughness properties decrease. Charpy V-notch impact (CVN) tests, pre-cracked CVN tests and other fracture toughness tests can identify the nil ductility temperature (NDT) - the temperature below which a material loses all ductility and fractures in a brittle manner. On a microscopic level, this equates to a change in the fracture mechanism from shear to cleavage. Fracture that occurs by cleavage at a nominal tensile stress below yield is referred to as a brittle fracture. A brittle fracture can occur in structural steel when a particular combination of low temperature, tensile stress, high strain rate and a metallurgical or mechanical notch is present.

Plastic deformation can only occur through shear stress. Shear stress is generated when uniaxial or bi-axial straining occurs. In tri-axial stress states, the maximum shear stress approaches zero as the principal stresses increase. When these stresses approach equality, a cleavage failure can occur. Welding and other sources of residual stresses in combination with yield level seismic generated stresses can set up a state of tri-axial stress leading to brittle fractures, if the connection is not properly detailed.

The necessity for minimum toughness requirements is not agreed to by all. There is also disagreement as to how much toughness should be required. The AWS Presidential Task Group recommended minimum weld metal toughness values of 20 ft-lb. at various temperatures, depending on the anticipated service conditions. For base metal, a toughness of 15 ft-lbs at a temperature of 70 than the material notch toughness, the crack will remain stable, and either elastic or plastic deformations will occur. Stress intensity factors greater than the material notch toughness indicate that brittle fracture is probable.

5.5.5 Temperature

Temperature and loading strain rate are variables that must be accounted for when determining notch toughness of a material. The relationship between notch toughness, temperature and strain rate is shown schematically in Figure 5-2. Typically, as temperature increases so does notch toughness and as the strain rate increases notch toughness decreases. This general statement is correct provided a lower transition temperature for notch toughness is exceeded. Similarly, the notch toughness increases until a limiting value is reached at some temperature and strain rate.

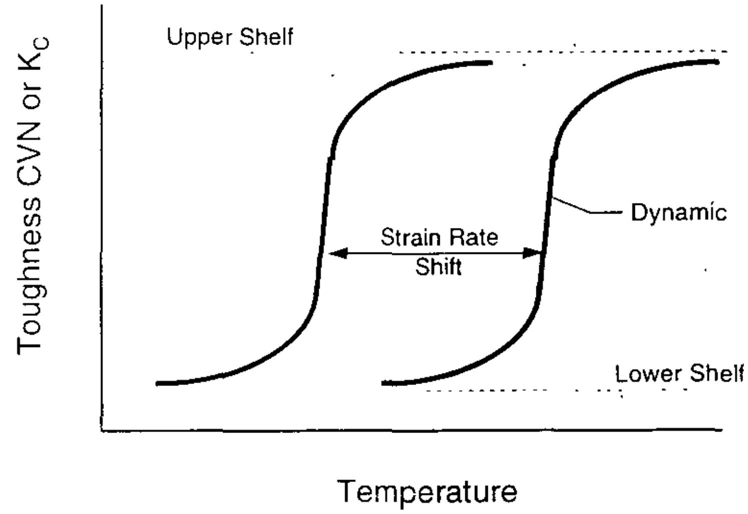

Figure 5-2 - Schematic Relationship Between Notch Toughness, Temperature and Strain Rate

5.5.6 Determining Notch Toughness

Over the years, numerous test methods have been developed to determine notch toughness. Many of these tests have been developed for specific purposes, others are more general but also more costly or difficult to perform. The Charpy V-notch (CVN) test fulfills several functions. Overall it is relatively inexpensive and therefore suitable for use as a quality control procedure. All specimens are identically manufactured with only the test temperature a variable. Provided reasonable care is exercised during production and testing, acceptable test repetitiveness can be accomplished Conversion of CVN data to dynamic notch toughness and hence to static notch toughness or some intermediate strain rate is done using an empirical relationship such as:

$$K_{ID} = \sqrt{5E(CVN)}\Big/_{1000}$$

the 20 years preceding the 1994 Northridge earthquake, the most commonly employed grade of FCAW-S wire was the American Welding Society (AWS) designation E70T-4 with properties specified in AWS A5.20: Carbon Steel Electrodes for Flux Cored Arc Welding. Tests of this product indicate CVN toughness values in the low single digits at 70°F can be expected. At this level of notch toughness the critical defect location is now in the weld metal and not the HAZ. Under these conditions, any weld root defect has the potential to become fracture critical and a potential source of brittle fracture initiation. Numerous examples extracted from Northridge earthquake damaged buildings confirm this scenario.

> *Commentary: The relationship between hydrogen level and notch toughness is not clearly identified in the literature and therefore there is no way to quantify the effects of hydrogen on notch toughness. Artificial aging of FCAW weld metal is not included in the AWS coupon preparation (AWS A5.20-95) for Charpy V-notch samples. Artificial aging of tensile coupons (permitted by AWS) tends to decrease hydrogen levels and increase ductility. Because deposited weld metal in WSMF connections is not artificially aged, the use of any FCAW-S filler metal that does not have a specified CVN values in AWS A5.20 and A5.29 should not be used. Until familiarity with a specific FCAW-S filler metal is developed, supplemental CVN testing of as-deposited weld metal in accordance with ASTM 673 may be appropriate*

5.6 Connections Conducive to Brittle Fracture

5.6.1 Loading Conditions

In typical welded, unreinforced beam-column joints, a critical state-of-stress occurs at the interface between the beam flange and the column flange under severe rotational loading of the connection. Such loading causes tensile stress in the beam flange and also produces tensile stress in the column flange. The same is true for compressive stress in the beam-flange to column-flange connection locations. The exact magnitude of the tensile stress in each flange is than dependent on the beam and column flange proportions. The vertical gravity stress on frame columns is usually not a significant factor because the columns are often sized for drift control under lateral load and not for live and dead load conditions.

Typically, for these connections, a plastic hinge is assumed to develop in the beam adjacent to the column under lateral loading. As a result, yield level stresses are expected to occur in the beam flange and large tensile stresses below yield are expected to occur in the column flange. These loading conditions produce a partially restrained stress condition with a high degree of tri-axial stress. Therefore, brittle fracture is a possible result in the presence of defects and low notch toughness material. Connections with base and weld metal, with adequate notch toughness, and the absence of rejectable notches or discontinuities will develop plastic flow (yielding) in the base metal adjacent to the beam-flange to column-flange weld and exhibit more ductile behavior.

5.6.2 Critical Connection Configurations

The loading condition and state-of-stress at the intersection of a beam and column has been described in the preceding section. Based on this information, various connection configurations can be described that are conducive to brittle fracture before adequate inelastic rotation can be sustained. The order in which they are listed generally, but not conclusively, reflect on ascending ability to deform inelastically.

1. Welded FR connections fabricated with low notch toughness weld metal, left-in-place backing bars and significant workmanship deficiencies.

2. Welded FR connections fabricated with low notch toughness weld metal, but with backing bars removed and with welds reinforced with large overlays of high toughness weld metal (Simon – 1997).

3. Welded FR connections fabricated using specified notch toughness base and weld metal and improved details and workmanship. Improved details include removal of backing bars and run-off tabs and incorporating large reinforcing fillet welds above and below the CJP. Continuous inspection from fit-up to weld completion to ensure strict compliance with an approved WPS.

4. Welded FR connections using reinforced beam-flange to column-flange details that result in plastic hinge formation away from the column face. The connection details and geometry are such that the column face weld stresses remain below the yield stress of the adjacent beam flange. This configuration can be accomplished using cover plates, vertical rib plates and several proprietary systems. In addition, the column-flange face stress levels equivalent to those produced by reinforcing plates can be achieved by the reduced beam section (RBS), or dogbone concept.

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TEMA

FAILURE ANALYSIS OF WELDED STEEL MOMENT FRAMES DAMAGED IN THE NORTHRIDGE EARTHQUAKE

EXPOSITOR: DR. DAVID DE LEON ESCOBEDO
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Failure Analysis of Welded Steel Moment Frames Damaged in the Northridge Earthquake

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ABSTRACT

A study was performed to characterize the origin of fracture and material properties of welded steel moment frame (WSMF) connections damaged in the Northridge earthquake. Sixteen connection fractures were obtained from five different buildings in the Los Angeles area which suffered damage in the earthquake. These fractures represented a variety of the types of fractures observed in post-earthquake building inspections. The mechanical and physical properties of the connection members and weld metal were determined including composition, strength, and fracture toughness. A fractographic examination of the fracture surfaces was performed to locate and characterize the fracture origin and determine the fracture mechanism. A fracture analysis was performed using linear elastic fracture mechanics. The analysis indicated that in all cases fracture resulted from crack instability which developed within the weld metal at the weld root at an incomplete fusion flaw contiguous with the notch introduced by the weld backing. The weld metal in all cases was determined to be E70T-4 weld metal and was found to have very poor fracture toughness. The fracture toughness of the weld metal was estimated to be 44 $\text{MPa}\sqrt{\text{m}}$ to 65 $\text{MPa}\sqrt{\text{m}}$ (40 $\text{ksi}\sqrt{\text{in}}$ to 60 $\text{ksi}\sqrt{\text{in}}$). A fracture mechanics analysis of the defect condition based upon the measured material properties and flaw sizes indicated that the cleavage fracture initiation observed in all the connections would occur without significant yielding in the beam flange and in some cases would occur under elastic stresses. Estimates of stress levels at the sample connections experienced during the earthquake were determined using simulated ground motion spectra for each building site and compared to the fracture analysis model. In all cases the range of estimated stresses exceeded the fracture stress predicted by the fracture model.

Keywords: Brittle failure; building technology; connections; earthquake damage; failures; fracture; frames; steel; structural failures; welded joints.

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1. Introduction

In the aftermath of the Northridge earthquake, extensive cracking was discovered in welded steel moment frame (WSMF) connections in more than one hundred buildings in the Los Angeles area. Speculation on the reasons for the fractures was widespread and focused on the external appearance of the cracked elements. Based upon the visual appearance of the cracking, a seemingly wide variety of types of fractures were observed and attributed to an equally wide variety of factors including the weld process, quality of workmanship, base metal properties, and connection design (Miller 1994; Campbell 1995; Tide et al. 1996).

This study was undertaken in order to characterize the origin of the fracture and material properties of failed connections thereby permitting a rational failure analysis to be made. The brittle nature of the weld joint fractures, which exhibited little evidence of inelastic behavior in the weldment or base metal, suggested that it was also desirable to ascertain the weld and base metal fracture toughness properties in addition to the traditional material properties of yield and tensile strength. With knowledge of the material fracture behavior and fracture initiation site, a fracture mechanics analysis could then be performed as a means of understanding the observed behavior of these connections (Fisher et al. 1995).

Sixteen connection fractures were obtained from five different buildings in the Los Angeles area which were damaged in the earthquake. The buildings selected were located at different distances and directions from the earthquake epicenter and each suffered a different extent of damage. The buildings were of sizes, designs, and ages representative of a large number of the buildings damaged in the Northridge earthquake.

1.1 Building Descriptions

The general layout of the five buildings is shown in the drawings in Appendix A. The buildings have been identified by an assigned letter (A, B, C, E, and F). The drawings include elevations of the lateral force resisting moment frames and show member sizes, story heights and locations where samples were taken. Plans of each floor are also included showing an outline of the building, locations of the moment frames, plan dimensions, and the locations of major openings and gravity columns. They do not include the layout of floor framing and slab reinforcing details. The building drawings also include a description of all damage to the moment frames sustained in the Northridge Earthquake. A brief description of each building follows.

Building A

Building A is a six story office building, designed circa 1980, with typical plan dimensions of 95 m (312 ft) by 49 m (160 ft). The plan area is approximately 3900 m^2 (42,000 ft^2). Typical story heights are 4.27 m (14 ft).

The building has multi-bay exterior moment frames on all four sides (column lines A, K, 1 and 5) and one interior frame in the transverse direction (column line E). All bay widths are 4.88 m (16 ft). Typical frame beams vary from W24x62 to W30x99 and frame columns vary from W14x99 to W14x211.

The floors are 82.55 mm (3.25 in.) of lightweight concrete over 76 mm (3 in.) metal deck. Floor beams and girders span 9.75 m (32 ft). The building rests on a pile foundation with pile caps. The bases of the frame columns are restrained against rotation by tie beams.

Building B

Building B is a four story office building with two levels of parking below ground. The building was designed circa 1984. Typical plan dimensions are 43 m (140 ft) by 26 m (86 ft). The typical plan area is approximately 1115 m^2 (12,000 ft^2). Typical story heights are 4.27 m (14 ft).

The building has single two bay exterior moment frames on Lines 4, A and G. There are a pair of two bay frames on Line 1. Bay widths vary from 4.88 m (16 ft) to 9.75 m (32 ft). Frame beams vary from W18x35 to W30x108. Frame columns vary from W14x48 to W14x211. Below grade, in the parking levels, the lateral load resisting system consists of concrete shear walls.

The floors are 82.55 mm (3.25 in.) of lightweight concrete over 76 mm (3 in.) metal deck. The floor framing is irregular but floor beams and girders typically span 9.14 m to 9.75 m (30 ft to 32 ft). The steel columns continue into the basement and rest on spread footings.

Building C

Building C is a four story office building that sits on a single level of parking. The lateral load resisting system for the parking level is reinforced concrete block shear walls. The frame columns terminate at the top of the shear walls. The building was designed circa 1983. Plan dimensions are 52 m (170 ft) by 30 m (98 ft). The plan area is approximately 1486 m^2 (16,000 ft^2). The first floor height is 4.72 m (15 ft 6 in) and the columns are pinned at the base. Typical floors are 4.04 m (13 ft 3 in).

The building has a combination of one and two bay exterior frames (Lines A, B, E, 1, 3, 4, 7). Bay widths range from 6.7 to 10 m (22 to 33 ft). Frame beam sizes range from W16x31 to W36x260 and frame columns from W14x68 to W14x500.

The floors are plywood sheathing over truss joists. Joists and girders typically span 6.7 m to 10 m (22 ft to 33 ft).

Building E

Building E is eleven stories tall with six stories of offices above five stories of above grade steel framed parking. The building was designed circa 1984. The plan is highly irregular. The plan dimensions in the parking levels are 79.25 m (260 ft) by 57.9 m (190 ft) (approximately 4550 m^2 (49,000 ft^2)). The office plan dimensions vary from 79.25 m (260 ft) by 57.9 m (190 ft) to 62.5 m (205 ft) by 44.2 m (145 ft). There is a 79 m long two story reinforced masonry shear wall at the south side of the structure that also serves as a lateral load resisting element. Typical story heights are 3.35 m (11 ft) in the parking levels and 4.27 m (14 ft) in the office levels.

The building has both interior and exterior multi-bay moment frames (Lines 2, 6, 9, 11, C, E, H, M). Typical frame beam sizes are W36x150 to W36x210. Large beams were used even in the highest levels. Typical columns sizes are W14x257 to W14x398.

The floors are 63.5 mm (2.5 in) of hardrock concrete over 76 mm (3 in) metal deck. Floor beams and girders span 8.84 m (29 ft). The building rests on a pile foundation with grade beams that fix the base of the columns against rotation.

Building F

Building F is a four story office building above three levels of below grade parking. The building was designed circa 1985. The plan dimensions are 44.5 m (146 ft) by 33.2 m (109 ft). The plan area is approximately 1486 m^2 (16,000 ft^2). Story heights range from 4.04 m (13 ft 3 in) to 4.72 m (15 ft 6 in) in the office levels and from 3.05 m to 3.73 m (10 ft to 12 ft 3 in) in the parking levels.

The building has four single bay exterior frames (Lines A, D, 1 and 6) with bay widths of approximately 12m (40 ft). Frame beams are W36x182 and W36x230 and frame columns are W14x257 and W14x311 sections.

The floors are 82.55 mm (3.25 in.) of light weight concrete over 76 mm (3 in.) deck. Floor beams and girders span 10.36 m to 14.02 m (34 to 46 ft). The building rests on strip footings.

2. Building Fracture Samples

Samples were taken from fractured connections in each building to allow characterization of the base metal, weld metal and fracture surfaces. In this section, results of visual inspection of the fractured connections and descriptions of the samples are presented along with the procedure for removing the samples from the buildings.

2.1 Sample Removal Procedure

A sketch of the WSMF fracture sample and removal procedure desired for the study is shown in Figure 1. The desired sample included the entire fracture, weld joint, and adjacent material for material property evaluation and fractographic examination. The sample is cut from the column flange above and below the weld joint and separated from the beam flange several inches back from the weld joint. This allowed the entire weld joint and fracture to be removed as a single unit.

This sample and removal procedure was ideal for the purposes of the study and was followed in the removal of a number of the fracture samples. However, it was not always adhered to depending upon the particular damage at a connection and the subsequent repair procedure followed. In some cases the column flange was removed in two or three pieces, particularly in connections with heavy columns. Apparently this procedure was followed to reduce the weight of the pieces being removed but unfortunately often damaged key areas of the fracture surface. Also, in separating the beam flange from the column, a cut was sometimes made along the weld joint, eliminating weld metal for mechanical property evaluation. Despite these difficulties many of the fracture samples removed were of good quality with little or no damage to the crack surface or adjacent material during removal.

2.2 Fracture Samples

Table 1 provides a summary of the sixteen connection samples removed from the five buildings including the sample identification (building letter identification and numeric code assigned to the connection), damage type and location, member sizes, and description of the sample removed. The location of the sample in the building can be found on the elevation drawings in Appendix A where the sample number corresponds to the circled connection number.

All sixteen samples were removed from bottom flange weld joints. This is partly due to the higher frequency of bottom flange fractures but also because they were more readily accessible for removal. The connection designs were all similar with full penetration groove welded flanges and bolted shear tabs. The samples consisted of a relatively wide variety of beam and column sizes. Beam sizes ranged from W24x76 to as large as W36x210 (Group 2). Column sizes ranged from W14x120 to W14x398 (Groups 2, 3, and 4).

Surveys of building damage after the Northridge earthquake categorized connection damage by fracture type based upon visual and ultrasonic examination (Youssef et al. 1995; Bonowitz 1995). The ten fracture types identified in these surveys are shown diagramatically in Figure 2. The test samples in this study included three types of connection fractures based upon the visual appearance of the fracture. The most frequently represented fracture type among the sixteen fracture samples was Type C3 (10 samples) which resulted in fracture which penetrated across the column flange. Four connection fracture samples also exhibited fracture of the column which formed a "divot" in the column flange (Type C2). Two of the connection samples fractured in the vicinity of the weld fusion line in the column flange (Type W4). These three types of connection fractures have been observed frequently and are representative of a large number of damaged connections.

Overall views of the sixteen failed connection samples are shown in Figures 3 through 23. In all samples the fractures appeared to be brittle with little evidence of plastic deformation of the connection. The crack path was clearly evident in several of the samples where the column flange had completely fractured (C1, C18, C19, E150, E226, F38) or had been removed in pieces (A6, A165, A254, B13). The cross-sectional views of the column flange resulting from arc gouging through the column flange and weld joint in several of the samples clearly showed the crack which in all samples appeared to originate at the weld root at the notch introduced by the weld backing. Sample B60 showed evidence of a divot pull-out of column flange material on both sides of the weld joint. Other samples, removed in whole, showed no obvious visual signs of fracture (A33, A287, E549). Damage in these connections apparently had been detected by NDE. It will be noted that several samples (A6, B4, B8, and F38) were incomplete and missing either the beam portion of the sample or the column flange. In the case of Samples B4 and B8 column material was not removed as part of the connection repair since the fractures occurred close to the column flange surface. Apparently the repair of connections A6 and F38 did not require removal of a section of the beam flange.

2.3 Weld Quality

From a visual inspection, the fabrication quality of the welds in all sixteen samples indicated a level of workmanship which was largely in accordance with AWS D1.1 (1994) and AISC (Manual 1989). Conditions which were less than ideal were observed, however, such as irregularly shaped web cope holes, poor weld tab fit-up, and excessively large cap pass weld beads resulting in acute weld toe geometries. Although not strictly acceptable conditions, they are not unusual conditions and appear to be typical of fabrication practices prior to the Northridge earthquake. Although weld tabs were not always ideally fitted, tabs fitted as end dams were not observed in any of the samples as has been observed in post-earthquake inspections of many buildings.

3. Material Properties

Mechanical properties of the beam and column were determined for each sample to identify the grade of steel. The tests also provided information on the strength and strength difference between the beam and column member of each connection for studying the effects of member strength on fracture susceptibility. Similarly, weld metal was analyzed for each sample to determine its composition and mechanical properties in order to identify electrode type.

3.1 Base Metal Mechanical Properties

Standard 0.505 in dia. tension specimens (ASTM A370 1996) were fabricated from the flange material at the test location specified in ASTM A673 (1995) (i.e., 1/6W) where possible. In some samples it was necessary to locate specimens closer to the flange tips in order to avoid damaging the fracture surface.

A summary of the tension test results is given in Table 2. The results show that the mechanical properties of all beam material tested satisfy the mechanical property requirements of ASTM A36 (1996) structural shapes. The yield strengths of all beam flanges were found to be below 345 MPa (50 ksi), and with the exception of Building C samples, were generally in the range of 262 MPa to 290 MPa (38 ksi to 42 ksi).

The column material in all samples tested satisfied the strength requirements of ASTM A36 (1996). The column material in Buildings A, and E, and F also satisfied the strength requirements of ASTM A572 (1994) Gr. 50 when the expected reduction in strength between the flange and the web properties measured for material certification was considered. (The yield strength measured in the flange can be as much as 10% to 15% less than that measured in the web.) Even considering this reduction, it is unlikely that the column material in Buildings B and C would satisfy Gr. 50 requirements. It is possible that the material in Buildings B and C satisfies ASTM A572 Gr.42 requirements, although the material was not specifically examined for these requirements.

Examining Table 2, one observes that for three of the buildings sampled (Bldgs. A, E, and F) the yield strength of the columns exceeded that of the beams, whereas the opposite was found in the other two buildings (Bldgs. B and C). Whether this was intended by design in the latter cases or whether an incorrect grade of steel was substituted is unclear. Nevertheless, similar fractures developed in both situations and therefore fracture of the connections cannot be attributed to material strength or strength differences alone.

Where material was available, Charpy V-Notch (CVN) specimens were also fabricated from column material. The specimens were taken from at the AISC (Manual 1989) test location for Group 4 and 5 shapes (intersection of the web and flange midway between the inside flange surface and the flange centerline). Specimens were tested in accordance with ASTM E23 (1996) over a range of test temperatures in the transition temperature range. Test results are tabulated in Table 3. CVN transition curves for each column sample are included in Appendix B. The average result of tests at the AISC location for Group 4 and 5 shapes at the required test temperature of 20 °C is also shown in Table 2. Although the test requirement only applies to the Group 4 columns from Buildings C, E and F, tests were also performed on the Group 3 column sizes in Buildings A and B for information since column members in these buildings fractured as well. With the exception of Building E, the columns from all buildings satisfied the AISC requirement of 27 J (20 ft-lbs) @ 20 °C (Manual 1989). Building E columns showed CVN toughness in the core region of 14 J to 22 J (10 ft-lb to 16 ft-lbs) @ 20 °C. It is noteworthy that columns of similar size in samples from Buildings C and E provided substantially different toughness in the core region yet both suffered similar fractures of the column flange. This suggests that the column fracture properties are not a primary factor in the connection failure.

3.2 Base Metal Composition and Microstructure

The chemical composition of the beam and column member from each of the samples is given in Table 4. These analyses, along with the mechanical property data, enable the identity of the grade of steel (ASTM A36, A572, etc.) to be determined and also provide an indication of the weldability of the material and whether this may have been a factor in the connection fracture.

The compositional requirements of ASTM A36 and A572 Gr. 50 structural shapes are given below for comparison:

Element	ASTM A36 (wt%)	ASTM A572 Gr. 50 (wt%)
C	0.26 Max.	0.23 Max.
Mn	No Req.	1.35 Max.
P	0.04 Max.	0.04 Max.
S	0.05 Max.	0.05 Max.
Si	0.04 Max.	0.40 Max.
Nb		0.005-0.05 (Type 1)
V		0.01-0.15 (Type 2)
Nb+V		0.02-0.15 (Type 3,Nb 0.05 Max.)

The principal compositional difference between A36 and A572 is the intentional addition of one or more microalloying elements (V and Nb). The analyses show that the beam material in all samples meets the chemical requirements of ASTM A36 but does not contain sufficient amounts of either microalloying element to satisfy the requirements of ASTM A572. Considering the strengths measured in the beams and the chemical compositions determined by analyses it can be concluded that all of the beam material satisfied the specification requirements of ASTM A36.

In contrast, the column material in Building A satisfied the compositional requirements of A572 in all but one sample (A287). The sample from building F (F38) also met the compositional requirements of A572 as well as one sample from Building E (E549). The remaining samples from Building E and all the samples from Buildings B and C did not satisfy A572 compositional requirements.

Examination of the composition and mechanical property results of all of the samples indicates that all of the beam samples (where beam samples were available) satisfied the specification requirements of ASTM A36. The column material from Building F and all but one column sample from Building A satisfied the specification requirements of A572 as well as one column sample from Building E. All of the column samples from Buildings B and C satisfied the specification requirements of ASTM 36.

The microstructure of the column flange and beam flange material was also obtained to verify the metallurgical condition of these materials. A ferritic-pearlitic microstructure was found in all column and beam members. This is the expected microstructure for as-rolled ASTM A36 or A572 material. Micrographs of the microstructures are included in Appendix C.

3.3 Weld Metal Composition

Where sufficient weld metal was available to perform analyses, the chemical composition of the weld metal was also determined. The results of the analyses are also given in Table 4. The results are similar in all eight samples tested and satisfy the compositional requirements of AWS E70T-4 weld metal. The chemical compositions are also typical for this type of weld metal. Required and typical compositions are shown below:

Element	AWS E70T-4 (wt%)	Typical (wt%)
C	Not Specified	0.24
Mn	1.75 Max.	0.55
P	0.04 Max.	0.008
S	0.03 Max.	0.008
Si	0.90 Max.	0.25
Al	1.80 Max.	1.38

3.4 Weld Metal Fracture Toughness

Charpy V-Notch test specimens were fabricated from weld metal in all samples where the weld metal was not damaged in the process of sample removal. Tests were performed at room temperature and at temperatures in the transition temperature range when a sufficient number of specimens could be fabricated from a sample. Test results are given in Table 5. A combined plot of the test results, shown in Figure 24, indicate a similar and very low level of weld metal toughness in all eight connection welds tested which is consistent with laboratory CVN tests of E70T-4 weld metal (Kaufmann et al. 1996; Xue et al. 1996). The room temperature toughness of the weld metal in all of the samples tested was in the range of 9 J to 20 J (7 ft-lbs to 15 ft-lbs) and reached an upper shelf toughness of 46 J to 83 J (34 ft-lbs to 61 ft-lbs) at 100 °C.

3.5 Material Property - Connection Fracture Correlations

Considering the range of beam and column member sizes included within the test sample and the similar types of fractures which developed in these members, there is clearly no correlation between beam or column size and fracture susceptibility or type of fracture of the connection. As noted earlier, susceptibility to fracture also appeared to be unrelated to strength or strength difference between beam and column members or fracture toughness of the column material. For example, identical column flange fractures developed in Building A in similar size columns with similar member strengths where the column flange fracture toughness varied from 28 J (21 ft-lbs)

@ 20 °C to as high as 123 J (91 ft-lbs). Similarly, fractures from Buildings C and E occurred at connections with similar size column members with very different fracture toughness (95 J to 102 J @ 20 °C vs. 14 J to 22 J @ 20 °C).

Member strengths or strength differences did not vary widely within the sixteen samples. Beam strengths were found to be less than, equal to, or greater than column strengths. No correlation between strength or strength difference and fracture susceptibility or type was noted.

Low weld metal notch toughness was found to be the only consistent factor in the connection fractures. Eight samples from which CVN specimens were tested all showed low notch toughness. The weld metal composition of these samples was found to be consistent with E70T-4 weld metal. The appearance of the weld deposit in the remaining eight samples was similar to these and suggests that they too were likely welded with an E70T-4 electrode.

4. Fractographic Examination

The fracture surfaces of the samples were analyzed to characterize the fractures and to determine the location of fracture origin. Additionally, fracture initiating root flaws were measured.

4.1 Fracture Origin

The fracture surfaces of the samples which were not completely separated were exposed by cooling the sample to low temperature in liquid nitrogen and loading the weld joint in a testing machine. The fracture origin was identified in all samples where the origin had not been destroyed in sample removal, and the size and source of the originating defect was recorded. The fracture origins were also examined with a scanning electron microscope (SEM) to obtain information concerning the fracture mechanism at fracture initiation. The fracture surfaces of the sixteen samples and SEM micrographs of selected crack origins are shown in Figures 25 through 57. Also indicated on the photographs is the location where the brittle fracture initiated. This can be determined from examination of the chevron patterns on the crack surface which point back to the fracture origin.

Cleavage fracture was found to be the mechanism of crack propagation in all of the samples regardless of crack propagation path (ie. Type C2, Type C3 or Type W4 fractures). Also, the location and source of fracture initiation was found to be the same in all of the samples regardless of the path of crack propagation. In all samples where a crack origin could be identified, brittle fracture initiated close to the mid-width of the weld (near the column web centerline) usually from a weld root incomplete fusion flaw in this area. A higher incidence of weld root incomplete fusion is expected in this area due to limited access for welding afforded by the weld access hole. In several samples, however, fracture was observed to initiate at the weld root at a location where no visually detectable incomplete fusion flaw was found. In one sample (A33) fracture was found to initiate at the weld root immediately next to a weld root flaw (see Figure 28).

SEM examination of the origin area of Samples A33, A254, A287, E226, and E549 all showed that cleavage fracture initiation occurred directly from the weld root defect (see Figures 29, 35, 39, 53, and 56). No evidence of other fracture mechanisms associated with stable crack growth processes, such as low cycle fatigue from prior load cycles or hydrogen cracking during fabrication, was observed at the crack origin.

Weld cross-sections such as those shown for Samples A33 (Figure 27) and Sample A254 (Figure 36) indicated that fracture initiated from the weld root within weld metal. This is reasonable since the notch tip introduced by the weld backing or any root incomplete fusion flaw must necessarily reside within weld metal and the weld metal is known from CVN tests to have very low toughness. Although in most welds the coarse grained heat-affected-zone (HAZ) possesses the lowest toughness in comparison to unaffected base metal or most weld metals, the low toughness of E70T-4 weld metal provides a ready material to initiate cleavage fracture. Once initiated the dynamic crack can propagate outside of the weld metal into the HAZ or unaffected base metal in response to the principal stresses and material toughness.

In order to provide further evidence of the location of fracture initiation, an EDS (Energy Dispersive Spectroscopy) analysis was performed on the crack surface at the crack origin on several of the fracture samples. The analysis is capable of providing a semi-quantitative compositional analysis of a small volume (several microns in size) of material by x-ray emission spectroscopy. The composition of the column base metal is similar to that of the weld metal with the exception of aluminum (Al) content. The aluminum content of A36 or A572 is negligible in comparison with E70T-4 weld metal (1.4 wt% to 1.8 wt%) and therefore provides a means of determining the type of material within which the crack resides at the point of initiation. Figure 59 shows EDS spectra obtained from E70T-4 weld metal crack surface, column flange base metal crack surface, and crack origin fracture surface from Sample A33. The position of a peak in the energy spectrum corresponds uniquely to the presence of a particular element and the amplitude of the peak is a function of concentration of that element and other factors. As expected, the peak corresponding to Al for weld metal is substantially greater than that for base metal. The crack surface at the fracture origin of Sample A33 shows a peak corresponding to Al which is intermediate between weld metal and base metal but certainly greater than would be found within the HAZ (i.e., base metal). Figure 60 shows similar spectra obtained for Samples A254 and A287. The spectral peak for Al in Sample A254 is stronger and very similar to that in weld metal. The peak in A287, however, is weaker and more like base metal in amplitude. Although not conclusive, the tests provide additional evidence that the fracture origin lies within weld metal and taken with other evidence supports the conclusion that the fracture initiated within the weld metal.

4.2 Initial Flaw Sizes

The depths of all fracture initiating root flaws, as well as the thickness of the weld backing, were measured. Table 6 provides a summary of the measurements. Also shown is the sum of the weld backing thickness and flaw depth which represents the effective flaw depth. A large range of flaw sizes (0.8 mm to 10 mm) which initiated fracture of connections was found within the test samples. This sugessts that the weld root flaw itself is not the primary reason for fracture but when coupled with the weld backing provides a large effective flaw depth of 10 mm to 20 mm. Figure 58 shows the distribution of fracture initiating effective flaw sizes within the limited test sample. The minimum effective flaw size is represented by the weld backing thickness (9.53 mm). The most frequent fracture initiating effective flaw size is seen to be between 10 mm to 13 mm (0.4 in to 0.5 in).

5. Fracture Analysis

The fracture toughness tests of weld metal removed from the connection samples, other damaged building connections (Tide et al. 1996) and weld metal deposited in laboratory tests (Kaufmann et al. 1996; Xue et al. 1996) have consistently shown that the E70T-4 weld metal, extensively used in WSMF connections prior to the Northridge earthquake, provides a low level of fracture toughness. This is consistent with the preceding fractographic observations that indicate that brittle fracture of all samples examined developed by crack instability from a crack-like flaw located within this low toughness weld metal. In this section, the static and dynamic fracture toughness of the E70T-4 weld metal are estimated and results of a fracture analysis are reported.

5.1 Weld Metal Fracture Toughness

Figure 61 shows a plot of the fracture toughness of E70T-4 weld metal compiled from the Charpy V-notch test data acquired from tests of weld metal removed from the connection samples, other building connections, and specimens from laboratory weldments. The plot shows the dynamic fracture toughness, K_{ID}, of the weld metal obtained by applying the following correlation (Barsom and Rolfe 1987)

$$\begin{aligned} K_{ID} &= \sqrt{0.64E(CVN)} \quad (\mathrm{MPa}\sqrt{m},\ \mathrm{MPa},\ J) \\ K_{ID} &= \sqrt{5E(CVN)} \quad (\mathrm{ksi}\sqrt{in},\ \mathrm{ksi},\ \mathrm{ft\text{-}lb}) \end{aligned} \tag{1}$$

to the existing Charpy V-Notch data for E70T-4 weld metal (CVN is the measured Charpy V-Notch energy and E is the modulus of elasticity). The test data shows that the dynamic fracture toughness, K_{ID}, of the weld metal is in the range of 27 MPa$\sqrt{m}$ to 44 MPa$\sqrt{m}$ (25 ksi$\sqrt{in}$ to 40 ksi$\sqrt{in}$) at room temperature and reaches an upper shelf toughness of 60 MPa$\sqrt{m}$ to 76 MPa$\sqrt{m}$ (55 ksi$\sqrt{in}$ to 70 ksi$\sqrt{in}$) at 100 °C. The band of data arises from the variability of toughness found in multipass welds and normal CVN test scatter. Experience has indicated that toughness estimated from CVN testing represents a lower bound of fracture toughness. Also plotted in

Figure 61 are the results of 25 mm thick weld metal compact tension tests (ASTM E399 1990) of E70T-4 weld metal welded in the laboratory and tested statically and at intermediate loading rates (0.5 sec.) similar to those developed during earthquakes (Xue et al. 1996). The static fracture toughness of the weld metal at room temperature was found to be in the range of 65 $\text{MPa}\sqrt{\text{m}}$ to 71 $\text{MPa}\sqrt{\text{m}}$ (60 $\text{ksi}\sqrt{\text{in}}$ to 65 $\text{ksi}\sqrt{\text{in}}$) and was reduced to 60 $\text{MPa}\sqrt{\text{m}}$ to 65 $\text{MPa}\sqrt{\text{m}}$ (55 $\text{ksi}\sqrt{\text{in}}$ to 60 $\text{ksi}\sqrt{\text{in}}$) at intermediate loading rates. The test results show that a reasonable estimate of the range of toughness that can be provided by E70T-4 weld metal at intermediate loading rates is 44 $\text{MPa}\sqrt{\text{m}}$ to 65 $\text{MPa}\sqrt{\text{m}}$ (40 $\text{ksi}\sqrt{\text{in}}$ to 60 $\text{ksi}\sqrt{\text{in}}$) at room temperature and is not appreciably affected by changes in temperature.

Tensile properties of E70T-4 weld metal, determined from laboratory weldments (Xue et al. 1996) show a 0.2% offset yield point of about 448 MPa (65 ksi). Hence, the strain rate shift between the estimated dynamic toughness, K_{ID}, and the static fracture toughness, K_I, is (Fisher et al. 1995; Barsom and Rolfe 1987)

$$\begin{aligned} T_s &= 215-1.5\sigma_y \approx 120\ ^\circ\text{F} \quad (^\circ\text{F, ksi}) \\ T_s &= 101-0.12\sigma_y \approx 47\ ^\circ\text{C} \quad (^\circ\text{C, MPa}) \end{aligned} \tag{2}$$

An examination of the static K_C tests plotted in Figure 61 shows reasonable agreement between the dynamic toughness band and the static K_C tests.

The intermediate strain rate tests provide a strain rate shift of about 27 °C or about 2/3 of the full static to dynamic shift. These results are consistent with the level of shift assumed to occur in structural steel systems.

5.2 Fracture Model

The fracture instability that developed from the weld root defect in each of the fracture samples examined can be modeled as a simple edge crack, as shown schematically in Figure 62. The stress intensity, K, at the crack tip can be expressed as (Barsom and Rolfe1987; Fisher et al. 1995)

$$K = 1.12\,\sigma\sqrt{\pi a_{eff}} \tag{3}$$

where the effective crack depth, a_{eff}, is represented by the sum of the weld backing thickness and the actual depth of the weld root flaw and σ is the applied stress. A simple edge crack condition is insured since the length of the backing bar lack of fusion is of the order of the beam flange width and large in comparison to the depth of the notch it introduces. Figure 63 shows a plot of applied stress, σ, as a function of flaw size using Eq.(3) and the expected range of fracture toughness for E70T-4 weld metal (i.e., K_C = 44 $\text{MPa}\sqrt{\text{m}}$ to 65 $\text{MPa}\sqrt{\text{m}}$ (40 $\text{ksi}\sqrt{\text{in}}$ to 60 $\text{ksi}\sqrt{\text{in}}$)). The plot shows the conditions of applied stress and flaw size under which crack instability will develop. For example, for flaw sizes on the order of backing bar thickness (9.5 mm, 0.375 in) crack instability can occur at applied stresses of 241 MPa to 345 MPa (35 ksi to which is in the range of the yield strength of A36 material. The critical stress is further reduced by the presence of weld root flaws. The analysis indicates that the cleavage fracture crack initiation which was observed in every welded connection sample would occur without significant yielding in the beam flange and in cases where a large weld root defect existed could occur under elastic stresses.

Progression of the dynamic expanding crack will be influenced by the principal stresses and variations in material toughness. The combination of tensile stresses in the beam flange and bending stresses in the column flange result in principal stresses that can direct the crack in a variety of directions. This may lead to the development of fractures which extend across the column flange, divot type fractures, or fractures which simply extend along the fusion line of the weld.

The critical applied stress is also influenced by other factors that will increase or decrease the fracture stress. All of the fractures have tended to initiate at the mid-width of the beam flange. This is due in part to the higher probability of weld root defects at this location and also because of the higher stresses at this location; the stress distribution across the beam flange is not uniform with higher stresses at the web-flange junction. Also, variations in weld procedure can introduce either tensile or compressive residual stress at the weld root which will tend to decrease or increase the applied stress at fracture.

6. Estimate of Stresses at Connections

The stress levels at each of the sample locations during the Northridge Earthquake were estimated to compare to the critical applied stresses calculated in the fracture analysis. While the estimates do not include weld residual stress or stress distribution effects, they provide an indication of the magnitude of the applied stress at each connection during the earthquake. A summary of the method and results are provided in the following sections.

6.1 Ground Motion Estimation

There were no actual ground motion records from the Northridge Earthquake at or near any of the buildings selected in this study. Therefore, the ground motion at each site during the Northridge Earthquake was simulated by two analytically derived spectra.

A common method of arriving at estimates of site-specific ground motion involves the application of attenuation relations. These relations are based on extensive regression analyses of a comprehensive database of earthquake records. More recent attenuation relations provide spectral as well as peak ground motion estimates at a given site.

The attenuation relationship utilized in this study, developed by Boore, Joyner and Fumal (1993), is based on a set of carefully selected accelerograms with well-known site characteristics. The attenuation functional is of the form:

$$\log Y = b_1 + b_2(M - 6) + b_3(M - 6)^2 + b_4 r + b_5 \log r + b_6 G_B + b_7 G_C \pm \sigma_{\log Y} \quad (4)$$

where,

$$r = \sqrt{d^2+h^2}$$

In this equation, Y is the ground motion parameter (in cm/s for response spectra and g for peak acceleration); the predictor variables are magnitude (M), distance and depth (d and h, in km), and site classification (G_B = 1 for class B and zero otherwise; G_C = 1 for class C and zero otherwise); $\sigma_{\log Y}$ represents the variance of the estimation. Coefficients b_1 to b_7 are functions of period of vibration. Site classes are defined in terms of average shear wave velocity in the upper 30 meters of soil and range from less than 180 m/s for site class D to greater than 750 m/s for class A. Plots and tables of the resulting spectra for the buildings studied are presented in Figure 64-67.

6.2 Structural Modeling Procedure

Linear-elastic models of the buildings were constructed using the structural analysis and design program ETABS 6.0 (Habibullah 1994). All floor diaphragms were assumed to be rigid and a rigid panel zone equal to 50% of the beam and column section depths was assumed. For the purpose of calculating masses, centers of gravity, and mass moments of inertia, the total story weight was assumed to be uniformly distributed over the floor. Typical floor weights, assumed to be equal to the total dead load, were in the range of eighty to ninety pounds per square foot of plan area. The buildings were fixed against translation and rotation at the top of shear walls where they exist and the frame columns were continued to their lowest level and pinned at the base. The spectra were input one direction at a time, oriented in the direction of the frames. The modal responses were combined using the complete quadratic combination (CQC) technique (Naeim et al. 1995) with 5% of critical damping.

For each building the first three building periods, total base shear, base shear as a percentage of weight and stresses in the beams at the location of each sample are presented. The beam bending stresses were determined at the column faces for dead plus live load, the mean spectrum, and the mean plus one standard deviation (mean+1σ) spectum. Full dead load plus a best estimate of actual live load was used. In all cases the dead plus live load stresses are insignificant when compared to the magnitude of the seismic stresses and the uncertainty in their estimate.

6.3 Analytical Results

Building A

The analytically determined building periods are presented in Table 7. For Building A the first periods in the transverse, longitudinal and torsional directions are 1.4, 1.28 and 0.93 seconds.

Comparison with the spectra for this site, shown in Figure 64, indicates that the peak ground accelerations are 0.27 g for the mean spectrum and 0.43 g for the mean+1σ spectrum. The base shears for the mean spectrum, shown in Table 8, are 15,692 kN (3528 kips) in the longitudinal and 14,051 kN (3159 kips) in the transverse directions. These correspond to 20.9% and 18.8% of the total weight of the building (% g). The base shears for the mean+1σ spectrum are 28,267 kN (6355 kips) (37.7%g) and 25,870 kN (5816 kips) (34.5%g) in the longitudinal and transverse directions respectively.

The bending stresses in the beams are presented in Table 9[1]. It can be seen that the Dead plus Live load stresses are all approximately 7 MPa to 14 MPa (1 ksi to 2 ksi). For the mean spectrum, stresses in the sampled joints range from 210 MPa (30.4 ksi) at A254 to 292 MPa (42.3 ksi) at A287. For the mean+1σ spectrum stresses vary from 381 MPa (55.3 ksi) at A254 to 541 MPa (78.5 ksi) at A287. Demand/capacity ratios, defined as calculated stress divided by measured yield stress may be found in Table 9 for all five buildings.

Building B

For Building B the first periods in the transverse, longitudinal and torsional directions are 1.5 s, 1.49 s and 0.88 s, respectively. The spectra for this site are shown in Figure 65. The peak ground accelerations are 0.23 g for the mean spectrum and 0.36 g for the mean+1σ spectrum. The base shears for the mean spectrum are 2,713 kN (610 kips) in the longitudinal and 2,700 kN (607 kips) in the transverse directions. These correspond to 15.4% and 15.3% of the total weight of the building. The base shears for the mean+1σ spectrum are 5,035 kN (1132 kips) (28.6%g) and 5,022 kN (1129 kips) (28.5%g) in the longitudinal and transverse directions, respectively.

The bending stresses in the beams are shown in Table 9. Dead plus live load stresses are all less than 7 MPa (1 ksi) except at B13 where it is 23 MPa (3.4 ksi). For the mean spectrum, stresses in the sampled joints range from 221 MPa (32.1 ksi) at B13 to 242 MPa (35.1 ksi) at B60. For the mean+1σ spectrum they vary from 378 MPa (54.8 ksi) at B8 to 416 MPa (60.4 ksi) at B13.

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[1] Locations of the samples may be found on the elevation drawings where they are indicated by their designation (e.g., A287) within a circle. They may also be located on the plans where all of the joints are numbered. Beam sizes may also be found on the elevations.

Building C

For Building C the first periods in the transverse, longitudinal and torsional directions are 1.58 s, 1.42 s and 1.15 s, respectively. The spectra for this site, which are the same as those used for Building F, are presented in Figure 66. The peak ground accelerations are 0.38 g for the mean spectrum and 0.64 g for the mean+1σ spectrum. The base shears for the mean spectrum are 4,955 kN (1114 kips) in the longitudinal and 4,288 kN (964 kips) in the transverse directions. These correspond to 29.1% and 25.2% of the total weight of the building. The base shears for the mean+1σ spectrum are 9,514 kN (2139 kips) (55.9%g) and 8,100 kN (1821 kips) (47.6%g) in the longitudinal and transverse directions respectively.

The bending stresses in the beams are presented in Table 9. Dead plus live load stresses are all less than 14 MPa (2 ksi). For the mean spectrum stresses in the sampled joints range from 248 MPa (35.9 ksi) at C13 to 358 MPa (51.9 ksi) at C19. For the mean+1σ spectrum they vary from 505 MPa (73.3 ksi) at C1 to 689 MPa (100 ksi) at C19. These beams are all at the first floor above the ground. The columns to which they are attached do not continue into the basement and are modeled as pinned at the base. The detail at the base plate is one that would normally be modeled as pinned for design purposes but undoubtedly provides some fixity. To the extent this happens the beam stresses would be partially mitigated and the actual stresses may have been somewhat lower than the calculated stresses.

Building E

For Building E the first three periods are 1.65 s, 1.56 s and 1.00 s. The first two mode shapes are not aligned with the building axes and hence these two periods represent modes coupled in the transverse and longitudinal directions. The third mode is torsion. The spectra for this site is shown in Figure 67. The peak ground accelerations are 0.17 g for the mean spectrum and 0.28 g for the mean+1σ spectrum. The base shears for the mean spectrum are 12,388 kN (2785 kips) in the longitudinal and 13,944 kN (3135 kips) in the transverse directions. These correspond to 7.9% and 8.8% of the total weight of the building. The base shears for the mean+1σ spectrum are 23,152 kN (5205 kips) (14.8%g) and 26,563 kN (5972 kips) (16.8%g) in the longitudinal and transverse directions respectively.

The bending stresses in the beams are shown in Table 9. Dead plus live load stresses are all less than 14 MPa (2 ksi). For the mean spectrum, stresses in the sampled joints range from 65 MPa (9.4 ksi) at E226 to 130 MPa (18.8 ksi) at E150. For the mean+1σ spectrum, they vary from 125 MPa (18.1 ksi) at E226 to 248 MPa (36 ksi) at E150. These stresses are low relative to the other buildings in the study. Similar results were also found in a study (Naeim et al. 1995).

Building F

For Building F the first three periods are 1.51 s, 1.54 s and 0.90 s. These correspond to modes in the transverse and longitudinal directions and torsion respectively. The spectra for this site, which are the same as those used for building C, are shown in Figure 66. The peak ground accelerations are 0.38 g for the mean spectrum and 0.64 g for the mean+1σ spectrum. The base shears for the mean spectrum are 6,441 kN (1448 kips) in the longitudinal and 6,570 kN (1477 kips) in the transverse directions. These correspond to 26.8% and 27.4% of the total weight of the building. The base shears for the mean+1σ spectrum are 12,352 kN (2777 kips) (51.5%g) and 12,668 kN (2848 kips) (52.8%g) in the longitudinal and transverse directions respectively.

The bending stresses are presented in Table 9. There was a single sample taken from this building (F38). The dead plus live load stress is 3 MPa (0.3 ksi). For the mean spectrum the stress is 149 MPa (21.6 ksi) and for the mean+1σ spectrum it is 296 MPa (43 ksi).

7. Summary and Conclusions

Results of tests conducted to characterize the mechanical properties and composition of the weld and base metals of samples removed from damaged buildings, a fracture analysis of the weld metal, and results of response spectra analyses providing estimates of critical applied stresses, lead to the following observations and conclusions:

1. Fractographic examination of sixteen WSMF connection fractures from five buildings have shown that the fractures in all cases resulted from crack instability which developed at the weld root of the beam flange-to-column flange groove weld. Cleavage fracture was found to initiate from an incomplete fusion flaw contiguous with the notch introduced by the weld backing in most cases, however, several fractures initiated directly from the notch introduced by the unfused weld backing. The fracture origin location was invariably near the mid-length of the weld near the centerline of the beam web.

2. The weld metal in eight samples tested was found to have very poor fracture toughness (7 J to 14 J @ 20 °C). Based upon the CVN impact toughness the static fracture toughness was estimated to be in the range of 44 MPa$\sqrt{m}$ to 65 MPa$\sqrt{m}$ (40 ksi$\sqrt{in}$ to 60 ksi$\sqrt{in}$). The measured toughness properties and chemical composition indicated that the weld joints in all sixteen samples were welded with an E70T-4 electrode.

3. The mechanical and physical properties of the beam and column material in all samples were in accordance with either ASTM A36 or A572 Gr. 50 steels. With the exception of one building, the fracture toughness of the column material was found to be well in excess of the AISC requirement of 27 J @ 20 °C in the core region. No correlation between member size or base material properties and occurrence or type of fracture which developed in the connection was found.

4. A fracture analysis of the defect condition based upon measured material properties and observed flaw sizes indicated that the cleavage fracture crack initiation that was observed in every welded connection sample would occur without significant yielding in the beam

flange and in some cases would occur under elastic stresses. Uncertainties in stress conditions in the vicinity of the flaw tip (applied and residual) and the magnitude of the strain rate effect prevent further refinement of the applied loads which resulted in fracture of the connections.

5. Estimates of beam flange stress levels during the earthquake at the sample connections were obtained using an analytically derived ground motion spectra for the building. With the exception of one building (Bldg. E) the analyses indicated a range of stress at connections which were of yield point magnitude and which exceeded the predicted range of fracture stress for the defect condition existing at the connection. Estimates of stresses at several sample connections in Building E were significantly lower in comparison to other buildings and well below yield levels. The range of predicted fracture stress at these connections were marginally higher than the calculated estimates.

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MOUDLO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

THE NORTHRIDGE FRACTURES: ARE WE LEARNING THE RIGHT LESSONS ?

EXPOSITOR: DR. DAVID DE LEON ESCOBEDO
PALACIO DE MINERIA
SEPTIEMBRE DEL 2000

THE NORTHRIDGE FRACTURES:

ARE WE LEARNING THE RIGHT LESSONS?

HARDY H. CAMPBELL III

Many seismic engineers have reached the wrong conclusions about the behavior of connections in the Northridge, Calif. earthquake. New requirements will escalate construction costs needlessly.

On Jan. 17, 1994, at 4:31 a.m., a tectonic trauma shattered steel connections as though they were made of glass, and at the same time fractured some seismic design assumptions.

Brittle fracture, the sudden rupture of metal without prior yielding, has plagued the engineering community for years—not because it is unpredictable or difficult to control, but because the same lessons are learned and forgotten over and over again.

The welded flange-bolted web connection typical of special moment-resisting frame (SMRF) design suffered multiple fracture modes. To date, more than 100 steel buildings have been inspected and found to be damaged; undoubtedly more will be discovered. Typical damage involved primary crack initiation at the juncture between the surface of the lower beam flange groove weld root pass (which penetrates into the backing bar) and the surface of the column flange.

This phenomenon was greeted by much of the structural community in California with surprise, bewilderment and alarm. The basis of seismic design, the eternal ductility of the SMRF connection, was shattered. I believe this assumption was unfounded, supported by optimistic research and a poor understanding of material science.

I want to express my complete satisfaction with the performance of these so-called "failed" connections. Despite a design that is almost fracture guaranteed, despite evidence of poor welding and inspection in many cases, despite an earthquake of historic proportions, not a single building collapsed nor was a life or limb lost as a result of these fractures. The first duty of a connection is to maintain structural integrity, and the Northridge connections withstood thousands of intense aftershocks. The failures occurred at several parking garages and highway overpasses, but not at structures with SMRF connections.

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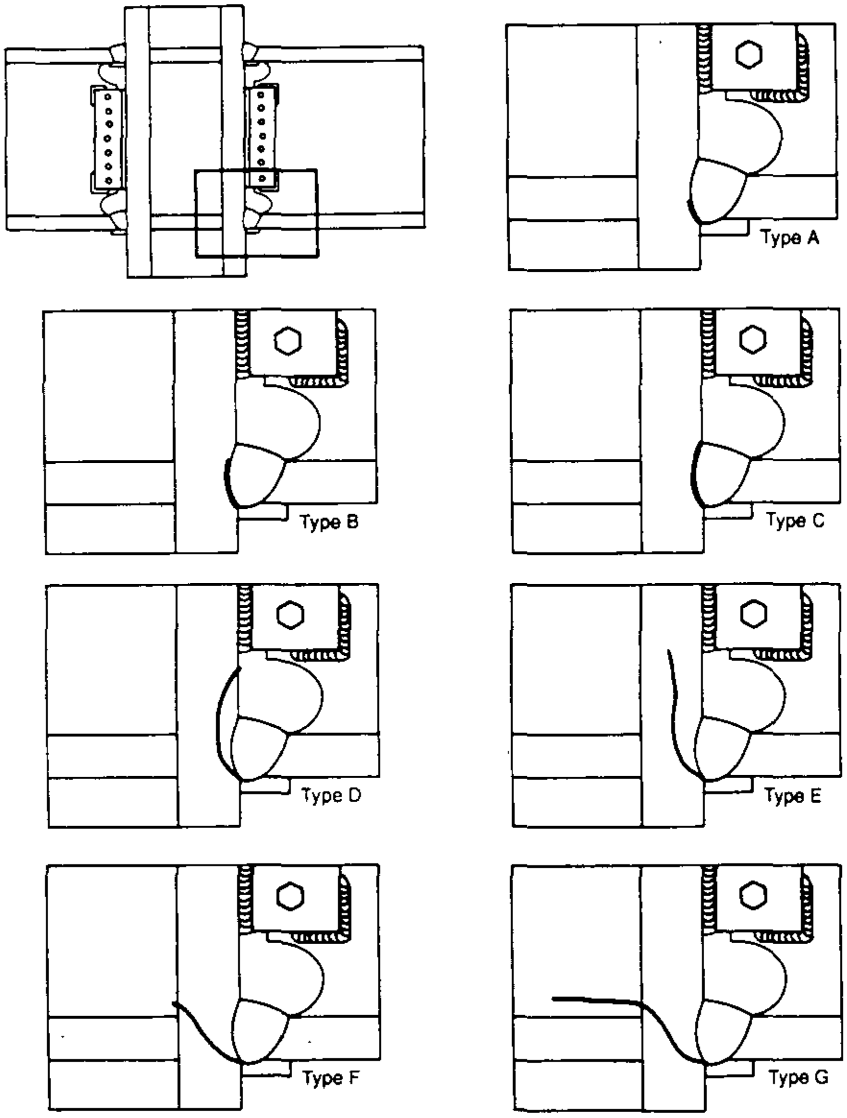
CRACK CATASTROPHE?

Cracks should be avoided, since they have the theoretical potential to propagate to a catastrophic loss of structural integrity Cracks can also significantly reduce load-carrying capacity. Yet neither possibility became reality in this event. Studies examining the effects on structural response may surprise some engineers Computer analyses of one damaged four-story structure looked at three cases, one with all flanges intact, another with both flanges detached and a third with only bottom flanges detached The intact structure had higher response loads and greater permanent drift than the two "damaged" structure cases. This suggests that Mother Nature knew what she was doing by fracturing these joints and making them semirigid. Perhaps there's a lesson to be learned here.

Cracks form because of a material's inability to accommodate stress through plastic flow or to absorb energy. Plastic deformation, the basis for ductile behavior, requires the movement of dislocations in a metal crystal lattice to arrange along slip planes; the easier this mobility is, the less stress is required to deform the material. The lowering of temperature reduces this mobility, and consequently increases the stress necessary to deform. Similarly, low temperatures reduce the energy required for fracture, and thus reduce the toughness of the material Toughness as measured by the Charpy V-notch test is expressed as fracture energy at temperature. Steel at very low temperatures can fracture with little or no application of stress, shattering like glass with the blow of a hammer

WELD CRACK TYPES FOUND AFTER THE NORTHRIDGE EARTHQUAKE.



TRIAXIALITY, STRAIN AND STRESS

However, temperature probably had little effect on the majority of damaged structures, since most were completed buildings covered with insulating materials. Still, for those unfinished, exposed buildings that suffered almost complete column fractures, the 40F early morning temperatures undoubtedly contributed to lowering fracture resistance. Two other factors with equivalent effects, triaxiality and strain rate, were present to lower fracture resistance (see Figs. 1 and 2).

Triaxiality is a condition by which deformation is restrained in all three directions. High triaxiality requires large stresses for plastic flow and reduces fracture energy, while fracture (ultimate) stress is relatively unaffected. This makes fracture an easier energy-dissipating mechanism than yielding. Thick flanges welded in rigid frames, like the Northridge connections, typically have high triaxiality. The residual stresses in the thick column flange that exacerbate triaxiality are a result of expansion and contraction induced by the mill production and field welding processes.

High triaxiality also degrades through-thickness properties, that is, the toughness, ductility and resistance to lamellar tearing of the column flange material normal to its surface Typically these values are considerably less than those measured in the longitudinal or transverse directions, even in the unrestrained condition.

Strain rates are a change in the load-induced strain as a function of time. Static loads are essentially constant; therefore, there is a zero strain rate. It follows that dynamic loads induce strain rates greater than zero. Since dislocations require time to arrange along slip planes, the higher the strain rate, the fewer mobilized dislocations per unit of time, the higher the yield stress and the lower the fracture energy.

A cohort of strain rate is the stress riser, which is a geometric transition in the stress flow path that magnifies the global (i.e., member design) stress into a higher local stress. Stress risers also magnify strain-rate effects, so that even a moderate global strain rate can, in the presence of a severe stress riser, have the effect of a high local strain rate.

At the root pass-column flange surface juncture where most of the cracks occurred, we find both primary culprits and their sidekick at work. If we examine this juncture closely, we find that the triaxial re-

2

straint is very high. We also see that the angle formed between the column and the weld at the juncture can vary considerably. a very acute angle (overlap) up to 90 deg (which is what the connection would have if it was carved from solid steel) and even slightly obtuse angles. These transitions constitute stress risers of varying magnitudes So at this juncture, the stresses induced by the first seismic shock were magnified to a high local stress, which added to the already high residual stress.

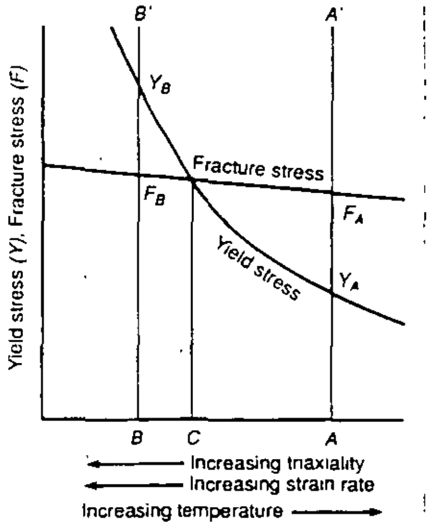
Because of the stress-riser–magnified strain-rate and triaxiality effects, the yield stress was higher than the nominal member yield stress and the fracture energy was very low. Though the combined local stresses were high, these were unable to achieve the high yield required at the juncture, but were adequate to reach the fracture stress.

THE WEAK LINK

This is not to say that juncture cracking was the exclusive fracture mode. In instances where gross weld defects occurred, such as joints with long lengths of unfused weld metal, the easiest fracture path was along the remaining fused joint, which was conse- tly undersized for strength and con- .d severe planar stress risers to degrade toughness Ultimately, the connection was doomed somewhere in the column-weld-beam chain; the seismic energy merely sought the weakest link to crack.

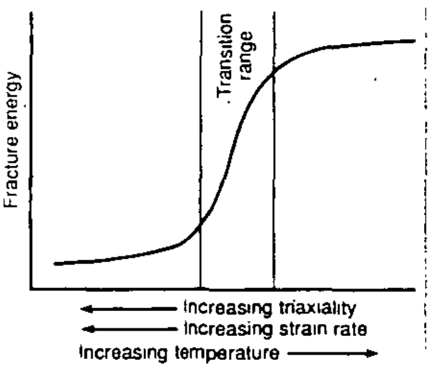
This analysis reveals some fundamental problems with the weak beam–strong column design philosophy That approach counts on the beams forming plastic hinges to dissipate the transmitted ground motion energy while the column accommodates severe rotations in a ductile fashion. A beam will typically have lower levels of triaxiality and lower magnitude stress risers than a connection So the fracture stress will always be higher than the yield stress, which will be the nominal member yield. This is essential for ductile behavior. However, for the connection, specifically at the juncture, the local yield stress Y_a is much higher than the fracture stress F_a. This is a characteristic feature of brittle fracture. For the same dynamic loading, as the beam tries to approach yield, the juncture must crack first. ed, no plastic hinge formation has been rted in any of the damaged structures. All of the Northridge fractures occurred well within the nominal elastic range of the beams and columns.

FIG. 1. CONNECTION STRESS



GENERALIZED VARIATION OF YIELD STRESS AND FRACTURE STRESS.

FIG. 2. DUCTILE-BRITTLE TRANSITION



GENERALIZED TRANSITION CURVE FOR DUCTILE-BRITTLE TRANSITION.

Engineers may have accepted the theory of weak beam–strong column due to the "success" of connection tests performed in the 1970s However, those tests used beam sizes considerably lighter than those used in SMRF design today, and thus minimized restraint. Also, those 1970s tests used very low (essentially static) strain-rate loading, presupposing that an earthquake would induce high stress, low cycle fatigue. This error in reasoning continues today. When several tests experienced failure, these were written off as an aberration.

Many engineers insist that dynamic-load effects were negligible in the Northridge event because the strain rates of a vibrating structure can be replicated with static tests. What they ignore is the embrittling nature of the initial, violent earth movement. The large spike in the ground motion accelerograms that typified Northridge cannot be viewed as static loading Many of the severe ground motions recorded (at one location, a maximum of 1.8*g* and 1.2*g* in the horizontal and vertical directions, respectively) approximate typical design values used for impact loading (1.0–2.0*g*). This is not a coincidence I have no doubt that the fractures occurred simultaneously with the initial ground jolt in classic brittle fracture fashion, and that fatigue played no part in crack initiation. This crack-initiating shock is independent of structural-response characteristics. It is purely a function of ground energy transmitted through the foundation and into the columns.

TESTING METHODS

Current testing methodology uses a slow reciprocating hydraulic piston to cycle connections to failure. This method does not represent the initial seismic conditions, so the results are unconservative. Even with this low-strain-rate application, the results of the tests performed to corroborate the adequacy of the modified designs have not been consistently acceptable. Additionally, the restraint inherent in actual frame connections (beams framing into both sides of a column node, beams restrained at each end) is not conservatively simulated by a cantilever welding on only one side of a column. Engineers would be prudent to accept these tests with healthy skepticism.

Though the standard connection design may be dead, it is reappearing with welded cover plates and vertical gussets. While these changes will undoubtedly reduce global stress, they will do little to reduce local stresses. The fractures were a local phenomenon, due to high triaxiality and stress risers, neither of which are significantly minimized in the new reinforcing schemes.

High strength frequently equals low ductility and poor toughness. Engineers are accustomed to seeking strength solutions suitable for static loads, instead of toughness solutions necessary for dynamic loads. This may be partly explained by the common design practice to treat seismic loads quasistatically to size members for strength. Northridge should alert them to dynamic material considerations.

Major rethinking is required in structural and connection design in order to withstand future earthquakes without significant fracture. Design optimization must seek to minimize demands on the inferior properties of material, and at the same time recognize the need for economical construction and adequate inspection. Structural design will have to dissipate energy more effectively, and this may require base isolation and semirigid connections that use deformable elements (such as framing angles) in the joint assembly.

For SMRF design structures, beam-column intersections must be detailed so that beam loads are not applied through the column flange thickness. One way to do this would be to have top and bottom horizontal continuity plates to which the beams could be field butt-welded. The column flanges would, in turn, be T-joint–welded in the shop to these plates, which are enhanced with through-thickness properties, using high toughness weld metal (column flange continuity would be provided by plates of beam depth). Many will argue that these measures are too expensive, but Northridge gave new meaning to the ounce of prevention.

Arriving at constructive solutions requires the acknowledgment of past errors. Many engineers have focused on what they perceive as the principal villain of this piece: shoddy construction. There is no question that this existed, but we cannot ignore deficiencies in the design itself.

The proliferation of stringent post-Northridge regulations pessimistically assess the connection's construction. This contrasts with the optimistic tests performed on the modified design. Neither can withstand close technical scrutiny. The SMRF connections would have fractured even if they had been carved from a block of steel, and the "new improved" connections will fracture just as surely.

Welding has always been the scapegoat whenever cracks occur, but in fact the majority of cracks occur due to poor design that does not recognize the fundamentals of material science. This discipline gets short shrift in most civil/structural engineering curricula. Many of the post-Northridge recommendations reflect a misunderstanding of material science, metallurgy, welding, construction and inspection, all vital in avoiding fracture problems. Minor issues such as peening are given inordinate attention while the major ones, such as design, are neglected.

SIX MYTHS ABOUT WELDING

Much of this response is due to some myths about welding, which I refute.

1. The welds cracked. In reality, most of the cracks began in the column base metal at the juncture. The relatively few weld-metal cracks observed were a result of secondary fractures from weld overload.

2. Self-shielded flux cored arc welding (FCAW-SS) produces crack-sensitive welds. In fact, FCAW-SS is the preferred process for field welding because of its relative insensitivity to wind compared to other processes and its history of quality production weldments. However, like everything else found deficient in the Northridge connections, noncompliance with the standard AWS D1.1 (from the American Welding Society) and a general lack of adequate quality control undermined its effectiveness.

3. The backing bars caused the cracks. All the bars did was move the root pass to below the groove root (located at the beam-column corner) into the bar itself so that the crack could initiate at the juncture. Much has been made of the "notch" between the unfused end of the bar and the flange being a crack promoter, but if this had been the case, cracking would have been randomly detected on either side of the notch. Cracks were only found on the column side, which supports the contention of the stress riser at the weld-column juncture. This stress exists regardless of the bar's presence, and has been acknowledged in new requirements to backgouge the groove root and finish with a fillet weld similar to the one required to reinforce dynamically loaded connections in D1.1. While this reduces the stress-riser characteristics, I believe it is of minimal benefit considering the severe loading requirements.

The first duty of a connection is to maintain structural integrity, and the Northridge connections withstood numerous intense aftershocks.

4. Filler metal notch toughness will prevent cracking. Merely requiring an arbitrary value of notch toughness for one link in the column weld-beam chain makes little sense. Indeed, the coarse-grained, heat-affected zone in the base metal traditionally has the lowest toughness, so why is the column base metal excluded from notch toughness requirements? The insidious part of this myth is that it is true to the extent that enhanced confidence in fracture resistance can be achieved with notch toughness, but only when design optimization minimizes the energy absorption demands on the material. Poor design has historically cracked the "notch-toughest" of materials.

5. Peening and postweld heating will prevent cracking. Similar to notch toughness, these postweld practices can offer enhanced confidence, but not in the presence of a poor design. These are expensive methods that are difficult to control and can frequently cause more harm than good when inexpertly applied. A good design can eliminate their marginal benefit altogether; a bad one won't be saved.

6. Making fabrication more expensive will prevent another Northridge. One fabricator told me how his prequake welding costs had exploded by 2,000% as a result of post-Northridge requirements. Unfortunately, plating connections with gold will only result in gold-plated connections with cracks in the next major earthquake.

I suggest that the input of expert technical organizations be sought prior to the imposition of such requirements. The AWS D1 Structural Welding Committee is carefully examining the provisions of D1.1 in an effort to determine how the use or misuse of the document could have contributed to the fractures. However, no code provision could have prevented a fracture-guaranteed connection from cracking. Nor can the code enforce compliance with its provisions; that is the responsibility of the client and contractor.

AWS wishes to cooperate with the efforts being made in California and elsewhere. What cannot be overemphasized is that these underdesigned connections survived a little brother of "The Big One" without a single casualty. ◆

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Coming next month in CIVIL ENGINEERING: another perspective on the steel-connections debate, as "Northridge Postscript: Lessons on Steel Connections" details what went wrong with steel connections in one seven-story office building featuring a welded-steel moment frame, how they were repaired and what lessons engineers can learn.

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XXVI CURSO INTERNACIONAL DE INGENIERIA SÍSMICA

MOUDLO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

NORTHRIDGE POSTSCRIPT: LESSON ON STEEL CONNECTIONS

EXPOSITOR: DR. DAVID DE LEON ESCOBEDO
PALACIO DE MINERIA
SEPTIEMBRE DEL 2000

NORTHRIDGE POSTSCRIPT: LESSONS ON STEEL CONNECTIONS

Mehdi S. Zarghamee
Rasko P. Ojdrovic

The rave reviews right after the Northridge, Calif. earthquake regarding the performance of steel-framed buildings have been replaced by concern following the discovery of flawed connections in more than 100 buildings. An investigation of one structure illustrates what went wrong with steel connections and how they were repaired.

In the immediate aftermath of the Northridge earthquake in California last January, the conventional wisdom was that concrete "crumbled like stale cake"—as one expert put it—and that steel was the preferred construction material in seismic regions. After the dust settled, however, investigators were surprised to discover the damage sustained by many welded steel moment-resisting-frame buildings.

These buildings had resisted past earthquakes remarkably well, but after Northridge, cracks were found in the moment connections of various structures. The first reported problems involved buildings either still under construction or very recently completed, such as the U.S. Borax building in Valencia (News, CE October 1994). Further inspection revealed weld cracks in more than 100 steel moment-resisting-frame buildings—75% of the buildings inspected. The potential for even more damage remains, because several hundred similar buildings have yet to be examined.

The cracks are usually concealed with fireproofing and finishes, and the buildings in which they have been found show only minor signs of distress. Although all the affected buildings are still standing, their resistance to future shocks may have been significantly compromised by the damage to their connections. The potential for catastrophic failure of damaged buildings during future earthquakes has prompted the profession to re-evaluate the connections, and has led the city of Los Angeles to draft an ordinance for the inspection of welded steel moment-resisting-frame buildings.

Simpson Gumpertz & Heger, Arlington, Mass., has undertaken several of these projects, including the inspection and repair of a seven-story office building with a welded steel moment frame. Our conclusions from this investigation hold important lessons for the engineering community as it addresses the condition of other steel-framed buildings in seismically vulnerable regions. Among the key findings are:

- Concealed connection failures that are not detected and repaired could compromise a building's resistance to future earthquakes. Our experience confirms the need for inspection of unbraced steel moment-frame buildings, as required by the Los Angeles ordinance, even though there may be no outward sign of significant damage to the building.
- Failure of moment connections may not be so prevalent that it would necessarily be discovered by an inspection of only a handful of connections. A large enough sample of connections should be inspected to help ensure that connection failures have been detected.
- The design of seated-beam simple connections typically used in steel frames, as recommended by the American Institute of Steel Construction (AISC), is not suitable for buildings subjected to earthquake loading with cyclic loads and significant joint rotations.
- Conventional welded moment connections lack the necessary ductility for the expected magnitude of plastic deformation of steel moment-resisting frames in a design earthquake and, as a result, are not suitable for buildings subjected to earthquake loads. Moment connections must force the plastic hinge to form at a location away from the beam-column interface.
- Ultrasonic testing (UT) of groove welds that connect beam flanges to columns does not reliably detect cracks in the welds, especially where the backup bar is in place, as well as in the zone near the beam and column webs.
- The jumbo steel sections typically used in fabrication of the columns of unbraced steel moment frames have a notch toughness that is not acceptable for buildings

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subjected to the shock loads that may be imparted by a strong earthquake.

- The flux-core arc-welding process used to field-fabricate moment connections of a steel-frame building may produce weld metals with relatively low notch toughness.

SEISMIC ANALYSIS

During the earthquake, the building suffered some damage to the glass cladding as well as very minor cracking of the interior finishes. Visual inspection of the finished interior and fireproofing of steel members and connections did not indicate any significant damage. As a result, very few engineers would have suspected damage of the steel framing.

As part of our curtain-wall investigation, we performed a seismic analysis, the results of which showed the need for inspection of the critical connections of the steel frame. We immediately identified two problems: failure of nonmoment seated-beam connections and cracking of the moment-connection welds.

The moment frame of the building consists of two tubular systems constructed at different times about 15 years ago. The frame has spans up to 32 ft that provide wide-open areas plus floor heights of 13 ft and 9.5 ft for office space and parking levels, respectively The moment beam-to-column connections are designed as typical AISC-recommended connections with welded flanges and bolted webs. The nonmoment beam-to-column connections at moment frames are typical seated-beam connections. Away from the moment frames are simple bolted-web connections.

We conducted a seismic analysis following the 1991 *Uniform Building Code* (UBC) and the 1991 National Earthquake Hazard Reduction Program (NEHRP) recommendations, and performed dynamic-response-spectrum analysis. The analysis consists of first performing a modal analysis, then using the response spectrum to compute the modal response contributions. The total structural response is obtained by combining individual modal contributions.

We developed a three-dimensional finite element model of the building structure for seismic analysis. The finite element model simulates the elastic behavior of all the moment frames; the nonmoment-frame beams and columns were not included in the analysis. The deformations from the elastic analysis were then amplified to account for plastic deformation of the connections and for the *P*-delta effect.

The maximum calculated interstory drift for the 500-year return design earthquake is 2.5 in. based on the UBC and 3.8 in. based on NEHRP guidelines; the maximum rotation of connections is 0.017 rad based on the UBC and 0.026 rad based on NEHRP recommendations. We used NEHRP-recommended provisions to develop repairs of the connections because they are the state-of-the-art method for analysis of buildings subjected to earthquakes, and can help predict the behavior of buildings in earthquakes more accurately.

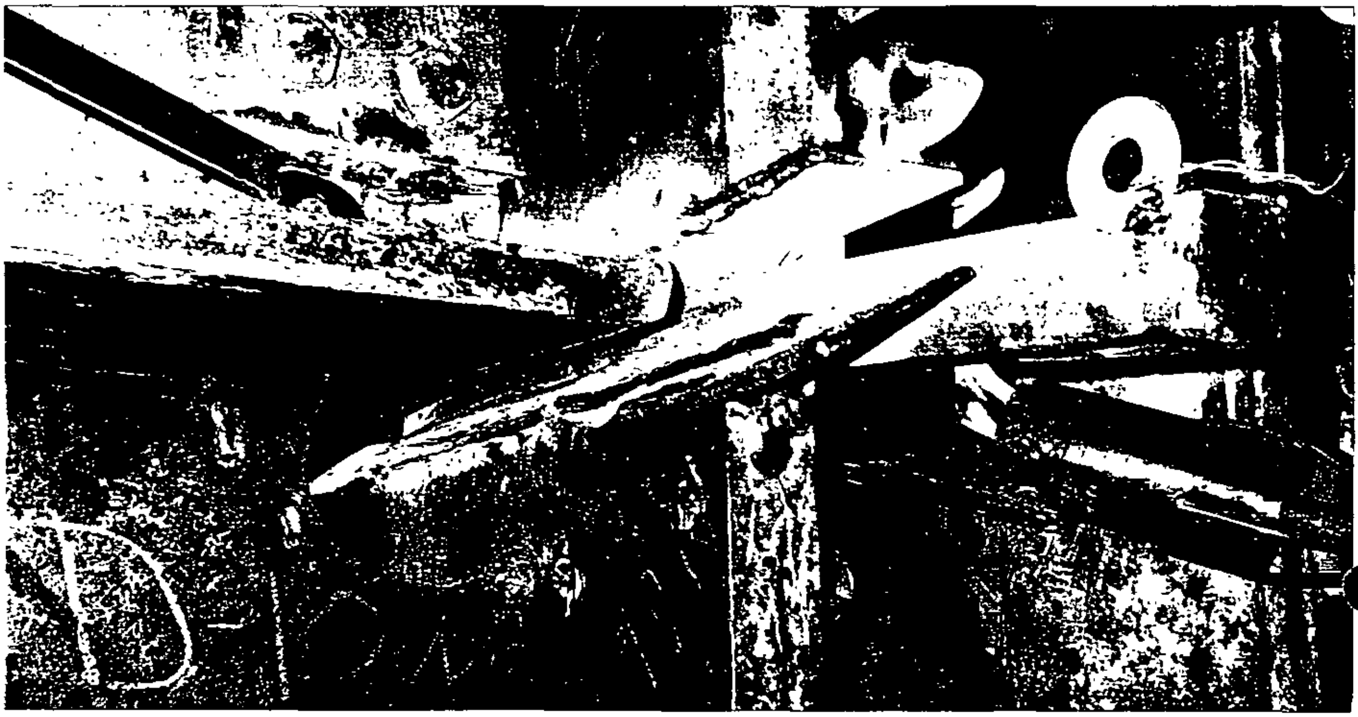
The Northridge earthquake had about one-half the acceleration of the design earthquake. Therefore, based on our deformation calculation, the building may have experienced an interstory drift of up to 1.9 in. and joint rotations of up to 0.013 rad. The magnitudes of drift and rotation are much larger than the values many engineers would estimate without doing the calculations. Large rotations coupled with large member sizes result in high strains imparted to the beam flanges of moment connections. The rotations also result in high strain rates when the building is subjected to an earthquake shock.

CONNECTION INSPECTION

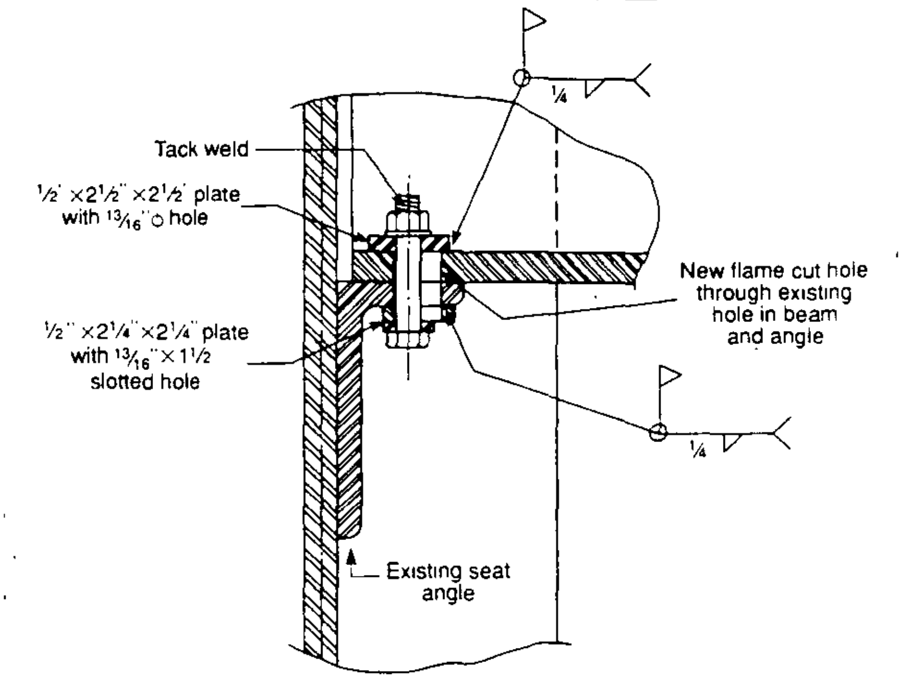
Based on the results of the seismic analysis, we recommended inspection of several steel framing connections at critical locations, especially those between the two tubular frame structures. After these connections were opened, we discovered that in most of the seated connections, many bolts connecting the beam flange to the seat were loose, sheared or missing. The building was then surveyed for plumbness and 10% of the moment connections were inspected and ultrasonically tested. Ultimately, all of the moment connections were inspected.

We performed a plumbness survey on

A CRACKED COLUMN FLANGE IS PREPARED FOR WELDING AFTER REMOVAL OF A CRACKED SECTION.



REPAIR SCHEME



several columns on three levels and found that the building had permanently deflect- the northeast direction. About 30% of rveyed columns were out of plumb by as much as 0.7 in. per floor level, higher than the allowable AISC erection tolerance of about 0.25 in. per floor

Inspectors also conducted visual and ultrasonic inspections of the moment connections. No sign of distress was detected visually; however, UT revealed cracks at or near the root pass of the groove welds connecting the bottom flange of the beam to the column flange. Cracks typically extended up through the weld or the interface between the weld and the column. We found that about 5% of all moment connections had large cracks penetrating across the entire weld width or into the column flange, and that about 30% had smaller defects and cracks. We decided to remove the backup bars from the bottom flange of all beams in moment connections in order to remove the natural notch created by the backup bars and clean any defects in the root pass.

The backup bar was removed by flame cutting, followed by grinding the cut surface oth to prepare it for magnetic-particle esting. Both visual and MP inspection aled numerous inclusions and cracks in the weld that were not detected in the initial ultrasonic tests. Most of the cracks were small and penetrated the column flange. Ironworkers ground out the inclusions and cracks until the MP inspection indicated sound material. We also observed scaling of the steel surface on the inside of most of the column flanges behind the bottom flange of the beam, indicating that the column has racked in alternating tension and compression, causing scaling in both flanges.

Several cracks spread during backup-bar removal. The central part of the column flange could not be inspected with UT because of the web-to-flange transition curvature. The pre-existing cracks and defects in this area may have extended across the flange width due to heating or moment redistribution. Almost all the observed cracks extended deeper into the flange closer to the column web. Some cracks extended across the flange width and into the column web. These cracks were immediately sted by drilling a 1 in. hole in the web e crack tip.

A significant finding during this phase of the project was that UT inspection with backup bars in place cannot reliably assess the condition of the weld's root pass. In particular, the weld and steel near the beam and column webs cannot be reliably inspected, and significantly more defects and cracks were found in this area upon backup-bar removal. UT inspection results depend, of course, on operator experience and technique

The columns with cracked flanges in this building are made of rolled jumbo shapes up to W14x370, with flanges up to $2^{11}/_{16}$ in. thick. The steel in the jumbo sections has a low fracture toughness, especially at the intersection of the flange and the web.

The welding process used for the field fabrication of many moment connections of steel-frame buildings is flux-core arc welding. Some self-shielded electrodes are high deoxidized types that may produce weld metals with relatively low notch toughness. In addition, the need for interpass slag removal and the high rate of deposition in this process produces high residual tensile stresses near the root of the groove welds and makes the welds highly prone to cracking and crack propagation in seismic shocks. Hydrogen migration into the weld, resulting from inadequate moisture control, can cause embrittlement and delayed cracking after completion of the weld. The practice of welding the bottom flange of the beam from both ends makes the central part of the beam weld susceptible to inclusions.

Backup bar removal demonstrated the shortcomings of UT inspection and of the welding procedure. UT inspection, intended to provide quality assurance, cannot reliably inspect the weld area that is most likely to have embedded defects and cracks. Such cracks, when subjected to an earthquake shock, can propagate through the weld and column flange, which as the lowest fracture toughness in the same area.

SEATED-BEAM CONNECTIONS

The AISC-recommended seated-beam connection is commonly used for nonmoment beam-to-column connections in unbraced moment frames of buildings designed to resist earthquake loads. This connection consists of a seat angle welded to a column and bolted to the bottom flange of the beam as well as a "flexible" top angle, welded to the column at the angle toe only and bolted or welded to the top flange of the beam, to provide stability.

Under gravity and live loads, the connection is designed to allow beam rotation through the flexibility and expected plastic deformation of the top angle. A test performed at Lehigh University, in which a flexible 4 by 4 by $\frac{1}{4}$ in. top angle was pulled out 1.98 in., demonstrated its flexibility. The connection is expected to behave as a simple support and accommodate positive (producing tension at top) and negative (producing tension at bottom) rotations as the building sways back and forth in an earthquake. As a result, the top angle is cyclically loaded in tension and compression.

To study the behavior of this connection subjected to a positive rotation, we repeated the test performed at Lehigh on the flexible top angle, but subjected the angle to a cyclic deformation of 0.62 in (the magnitude expected in the design earthquake) with joint rotation of 0.026 rad and depth of beam of 24 in. During our test, the toe of the weld started cracking after only a couple of cycles and failed after about 10 cycles. The weld returns proved useless when subjected to cyclic loading, indicating that the flexible welded top angles do not possess the required ductility when subjected to cyclic loading

When the connection was subjected to negative rotation, the joint pivoted about the top angle and resulted in significant lateral displacement of the bottom flange. Since the seated-beam connection cannot accommodate 0.62 in of relative motion between the seat and the beam flange, the connection fails by shearing of the bolts at the seat

We observed that the bolts connecting the beam to the seat and some of the top angles were sometimes deformed or sheared, but there was no sign of distress or damage in the seat angles and their welded connections to the columns. Deformation and shearing of bolts was caused by building motion during the earthquake as well as insufficient clearance in the bolt holes to accommodate the building motion. The bolts in the connections that have been deformed and sheared are not load-carrying bolts; these bolts stabilize the beams laterally during an earthquake. For gravity loads, the beams are safely seated on the angles welded on the columns. When the top-angle connection fails, due to either failure of the angle-to-column weld or failure of bolts, the beam remains seated, but the lateral movement of the beam is not reliably restrained

Our investigation shows that AISC-recommended seated-beam connections in unbraced steel frames are not suitable for buildings subjected to earthquake loads. The connection should be designed to enable beam rotation relative to the column for negative moments that may arise in an earthquake while it preserves lateral stability of the beam

When designing remedial repairs for the building, we used the joint rotations calculated in the dynamic analysis. The repair scheme is shown in the figure. The seated-beam connection has to allow relative rotation between the beam and the column. For positive rotation, the flexibility of the top angle allows the rotation to occur. The remedial design must allow a negative rotation to occur. Our design consists of burning holes in the seat angle and bottom flange, creating a slotted hole and thus allowing the negative rotation of the beam to occur freely. The bolt is offset from the center of the new hole (see figure). Since the condition of the top angle could not have been ascertained without demolition of the concrete deck, we designed two angles to provide lateral restraint to the beam web without interfering with the beam rotation. The angles are welded to the column only and are not welded to the beam.

MOMENT CONNECTIONS

The moment connections are intended to provide full transfer of moment and shear from a beam to a column. In a typical welded moment connection, this is accomplished by welding the beam flanges with full-penetration groove welds to the column flange and by bolting the beam web to a shear plate welded to the column flange. The design concept of strong column–weak beam assumes that, in an earthquake, energy is dissipated in inelastic deformation of the beam. In an unbraced moment frame with such moment connections, energy dissipation occurs at the connections. As demonstrated by the Northridge earthquake, these connections are frequently incapable of accommodating the necessary magnitude of plastic deformation when subjected to seismic loads without fracturing.

The tests conducted recently on large beam-to-column connections at the University of Texas at Austin shed light on the behavior of such connections. (The absence of concrete deck, lack of axial load in test columns and the slow rates of loading used in these tests have raised questions about the validity of the results when applied to the real case of a moment connection being subjected to a seismic shock.) The test results indicated that a typical moment connection could not sustain rotations greater than 0.009 rad, and three of four tested connections failed at much smaller rotations. Connections reinforced with beam-cover plates near the column flanges performed much better, and are potentially capable of sustaining plastic rotation of at least 0.025 rad. Official guidance for repairing and strengthening existing connections as well as designing new ones is not yet available.

The beam section near the connection to the column is inherently weaker than the rest of the beam, because of the flexibility of the beam-web connection to the column. Reinforcing a connection with cover plates reduces the tensile stress in the weld and the column flange. In addition, it moves the highest-stressed region away from the connection along the beam, where stresses are lower, material is more ductile, stress raisers do not exist and there is much smaller variation in material quality than in the weld zone. Consequently, the likelihood of plastic deformation is increased and the propensity to cracking is reduced.

The repair of moment connections with smaller cracks was performed by grinding out the defects and welding the ground-out material and finish with a fillet weld. The repair of connections with large cracks extending into the column flanges was performed by first welding a knee brace and then cutting the defective material and welding the cutout. The knee brace was added to transfer the beam moment due to gravity load into the column and to prevent joint rotation while the weld and the column flange were cut out. The knee brace was designed to transfer the moment and provide as much room as possible for the workers.

Repair of the connections was arduous. First, the connection was preheated in preparation for cutting the metal. Flame cutting was done to remove the weld connecting the bottom flange of the beam to the column flange. Then, workers removed a trapezoidal section of the column flange containing the crack. The cut above the bottom flange of the beam was horizontal, and the cut below was inclined so that all cracked material was removed; as a result, there was a clear opening of up to 2 in. on the inside of the column flange, and up to 3.25 in. on the outside of the column flange. After the cut surfaces had been ground smooth, a brass backup plate was attached to the inside of the flange. The connection was preheated and welding was performed using low hydrogen E-70-18 electrodes. The connections were successfully ultrasonically tested 48 hours after the repair work was completed.

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XXVI CURSO INTERNACIONAL DE INGENIERIA SÍSMICA

MOUDLO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

DISEÑO Y COMPORTAMIENTO SISMICO DE ESTRUCTURAS DE CONCRETO REFORZADO

EXPOSITOR: DR. OSCAR LOPEZ BATIZ
PALACIO DE MINERIA
SEPTIEMBRE DEL 2000

ESTAS GUÍAS PARA DISEÑO CONSIDERANDO LAS RESISTENCIAS ÚLTIMAS DE LOS ELEMENTOS ESTRUCTURALES CONTIENE TODOS LOS CONCEPTOS PRESENTADOS DURANTE LA SESIÓN.

Oscar López B.
Sept. 1999.

A I J DESIGN GUIDELINES
FOR
EARTHQUAKE RESISTANT
REINFORCED CONCRETE BUIDINGS
BASED ON
ULTIMATE STRENGTH CONCEPT
(1990)

PREFACE

The "AIJ Standard for Structural Calculation of Reinforced Concrete Structures" (hereinafter abbreviated as the AIJ Standard) was first published by the Architectural Institute of Japan (AIJ) in 1933 followed by several revisions, and it has been widely used as a method for the structural calculation of reinforced concrete structures in Japan The AIJ Standard, although basically based on the allowable stress concept when determining the safety criteria, is partially based on the ultimate strength concept by introducing a concept of higher allowable stresses for short-term working loads than those for long-term working loads, and the ultimate shear strength for shear design of columns and beams. Recently the importance of structural design based on the properties at the ultimate stage has been widely recognized. Hence, the Building Standard Law Enforcement Order, revised in 1980, adopted the requirement to examine the lateral load carrying capacity of a structure in its seismic provisions.

The design guidelines presented hereby requires a minimum lateral load carrying capacity of a structure to limit the response deformation during an earthquake, and the formation of a ductile total yield mechanism to dissipate earthquake input energy. The special feature of the design guidelines is that a structural designer should plan a desirable yield mechanism for a structure under an earthquake excitation and should design the structure to develop the planned yield mechanism during a strong earthquake In the first step of the design method (designated as the yield mechanism design), the designer should plan a desirable yield mechanism, and give both the required strength to the structure and sufficient ductility to the planned yield hinges In the second step (designated as the yield mechanism assuring design), the designer should provide non-yielding regions and members with sufficient strength to assure the formation of the planned yield mechanism of the structure. Another feature is a new proposal in shear design of members based on a plasticity theorem, in which shear is assumed to be resisted by concrete arch and truss mechanisms The unified shear design method can be used for beams, columns and structural walls

The earthquake resistance of the design method relies on the energy dissipation capacity at the planned yield hinges. Thus, the application of this method is limited to those structures that can develop a clearly defined yield mechanism. From this view point, the proposed design guidelines is not intended to replace the AIJ Standard that can be employed for various types of structures, but supplement the use of the AIJ Standard.

October 1990

Architectural Institute of Japan

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Design Example I (6-story building)	Working Group for Design Examples
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CHAPTER 1 . GENERAL REQUIREMENTS

1.1 Scope

The present Design Guidelines describes an earthquake resistant design method based on ultimate strength concept for reinforced concrete building structures, which shall form a yielding mechanism during a strong earthquake The buildings shall satisfy the following conditions:

(a) A moment-resisting frame structure or a moment-resisting frame structure with structural walls of regular configuration, and

(b) A total height of a building not more than 45 m.

[Commentary]

The Design Guidelines describes an earthquake resistant design method based on ultimate strength concept for a regular moment-resisting frame structure or a moment-resisting frame structure with structural walls of a total height not more than 45 m.

"The AIJ Standard for Structural Calculation of Reinforced Concrete Structures" is principally based on the allowable stress design procedure, and adopts partially ultimate strength design concept.

On the other hand, the Building Standard Law Enforcement Order, revised in 1981, use two-step procedures in earthquake resistant design: (a) The first step based on allowable stress design procedure; and (b) the second step to examine the ultimate lateral load carrying capacity of a structure and member ductility. The AIJ Standard is used mainly in the first step. The adoption of the two-step procedure is believed to have improved the earthquake resistance of a reinforced concrete building, however, because the second procedure is only an examination, a structural engineer is hard to envision the behavior of the structure at the ultimate state during the first step design procedure.

The Design Guidelines supplements the weakness of the AIJ Standard not to cover the second step design procedure, and furthermore shows a single-step ultimate strength earthquake resistant design method.

The philosophy in the Design Guidelines is believed to be commonly applicable to the design of general reinforced concrete buildings. However, the application is limited to regular buildings, which can develop a clearly defined yield mechanism under lateral loading. The limitation became necessary partially because of the shortage of analytical evidences and theoretical support. A building under design must be planned to have a uniform distribution of mass and stiffness along the height and in the plan so that a uniform response should be developed during a strong earthquake Unbalanced distribution of mass and stiffness is likely to cause a magnification of response in limited locations, resulting in the formation of partial yield mechanisms with concentrated damages in the locations.

Two reasons for the height limitation are given below.

(1) The fundamental vibration mode is dominant in a building whose total height is not too high. The Design Guidelines assumes the formation of a planned yield mechanism under fundamental mode vibration.

(2) Ministry of Construction requires a appraisal of design for a building of more than 45 m in height.

The methods specified in each chapter are still tentative due to the lack of understanding. This Design Guidelines, including the scope, will be reviewed with increase of research findings. In the practical application of this Design Guidelines, it is hoped that all users understand the concept behind it, and use it correctly.

1.2 Definitions

The following terms are defined for general use in the guidelines

Yield Mechanism : A mechanism formed by developing yield hinges in a structure subjected to horizontal earthquake forces
Frame Structure : A structure consisting of beams and columns which resist the actions caused by earthquake forces, commonly known as a moment-resisting frame.
Wall-frame Structure : A structure consisting of an independent multi-story continuous structural walls or of multi-story continuous structural walls within a frame structure.
Yield Hinge (Region) : Location (region) in a member where plastic deformation by bending moment is allowed to develop
Yield Mechanism Design or Mechanism Design : A design procedure to plan a target yield mechanism and to determine the resistance at planned yield hinges to satisfy the required lateral resistance
Yield Mechanism Assuring Design or Assurance Design : A design procedure to provide members with resistances necessary to avoid failure in region other than the planned yield hinges even under upper bound actions by an earthquake motion.
Overall Yield Mechanism : A yield mechanism in which uniform inter-story drift angles by plastic deformation are developed over the structural height, typically a mechanism formed by yielding at all beam ends and at the first story column base
Partial Yield Mechanism : A yield mechanism in which inter-story drift angles by plastic deformation concentrate in a limited number of stories.
Design Limit Deflection : Maximum inelastic response deflection of a structure or members allowed under a large intensity earthquake motion considered in design.
Assurance Deflection : Deformation capacity to which the design assures a structure and its members to perform without failure
Linear Analysis : A structural analysis under design earthquake loads assuming equivalent stiffness of members realistically evaluated at the yielding of a structure.
Nonlinear Analysis : A structural analysis under fundamental distribution of earthquake loads assuming upper bound flexural strength at the planned yield hinges
Primary Distribution of Earthquake Loads : Average distribution of horizontal loads expected to act on a structure during a strong earthquake.
Moment Redistribution : Modification of moments obtained by the linear analysis by satisfying the equilibrium of internal actions and external earthquake loads, for the purpose of rational and economical design and construction
Dynamic Effects : Magnification of member actions obtained by the static nonlinear analysis under the primary distribution of earthquake loads to take into account the contribution of higher mode effects
Concurrency of Bi-directional Earthquake Loads : Effect of simultaneous earthquake actions along both principal axes of the structure.
Ultimate Strength : Strength of sections and members in general.
Reliable Strength : Nominal strength of a section or member calculated as the lower bound, using section dimensions and minimum specified material strengths and taking into consideration the reliability of strength evaluation methods.
Upper Bound Strength : Nominal strength of a section or member calculated as the upper bound, taking into consideration all possible factors which contribute to the strength such as higher than specified material strengths, additional steel placed for construction purpose, reliability of strength evaluation methods, spread of effective width of slabs and orthogonal walls
Flexural Strength : Bending resistance evaluated for a section, neglecting the interaction by shear resistance.
Shear Strength : Shear resistance evaluated for a member assuming infinitely large flexural strength
Truss Mechanism : A shear resistance mechanism formed by longitudinal reinforcement of varying stress distribution, shear reinforcement, and inclined concrete strut.
Arch Mechanism : A shear resistance mechanism formed by longitudinal reinforcement of uniform stress distribution and a diagonal concrete strut between the two member ends.
Effectiveness Factor for Compressive Strength of Concrete : Reduction factor for compressive strength of concrete used in the evaluation of shear strength.
Special Yield Hinge : A yield hinge of a column or structural wall that requires a special reinforcement detailing due to high axial stresses
Non-structural Member : A member of a structure that is not planned to carry earthquake actions

[Commentary]

Representative terms are defined in this section.

1.3 Symbols

A_c	cross-sectional area of a column;
A_{ce}	effective cross-sectional area of a boundary column in a structural wall for shear strength;
A_{core}	cross-sectional area of a boundary column in a structural wall;
A_g	gross cross-sectional area of longitudinal reinforcing bars in column section;

A_v :cross-sectional area of a set of shear reinforcing bars;
A_{ws} :cross-sectional area of vertical reinforcing column in a wall panel,
A_{wx} .cross-sectional area of inclined reinforcing bars in a wall panel,
A_x :cross-sectional area of inclined reinforcing bars for shear strength;
C_1 :design base shear coefficient,
C_{1o} :base shear coefficient in the yield mechanism assuring design,
C_B :standard base shear coefficient,
D :overall depth of a section of beam or column,
D_c .overall depth of a section of a boundary column in a structural wall,
D_j :effective depth of a beam-column connection (column depth or horizontally projected length of 90 degrees hook),
F_c :design nominal strength of concrete,
F_i :horizontal load at the (i+1) floor;
H_i :height of the structure from the ground to the (i+1) floor level;
H_n :total height of the structure from the ground level;
L :clear span of a column or beam;
N_c :axial load of a column in the yield mechanism assuring design;
N_{cc} axial load of a boundary column in a structural wall;
N_w .axial load of a wall in the yield mechanism assuring design,
P_i :horizontal load at the (i+1) floor defined by a reverse triangular distribution of seismic intensity;
P_t concentrated horizontal load at the roof floor;
Q_1 :design base shear,
R_p .rotational angle at the yield hinge region associated with assuring deformation of a structural member;
R_s :yield mechanism assurance inter-story deflection;
R_t :vibration characteristic factor,
R_u :assuring rotational angle of a wall,
T :fundamental period of the structure;
V_j :design shear for a beam-column connection;
V_{ju} :reliable shear strength of a beam-column connection;
V_u :reliable shear strength of a beam or column;
W_i :sum of dead and live loads for earthquake loading at the (i+1) floor;
Z :earthquake zone factor;
b :width of a section of a beam or column,
b_{ai} :difference of dimensions of beam width and column depth;
b_b beam width;
b_i :distance from the face of a beam to the face of a column,
b_j :effective width of a beam-column connection;
d :effective depth of a beam or column;
d_b :nominal diameter of a longitudinal reinforcing bar;
h_s :height of spandrel beam or width of wing wall;
h_w :height of a wall for shear design;

j .distance from the centroid of the compressive resultant to the centroid of the tensile resultant;
j_t :distance between top and bottom bars in a beam or column;
k_1 :coefficient for compressive axial load limit of a column,
k_2 :coefficient for tensile axial load limit of a column;
k_3 :coefficient for compressive axial load limit of a wall;
l :span length of a beam or height of a column;
l_w :distance between centers of boundary columns within a wall;
$l_{w'}$:clear span of a wall panel,
l_{wa} :equivalent width of a wall panel in the arch (strut) mechanism;
l_{wb} :equivalent width of a wall panel in the truss mechanism;
n :number of stories,
p_s :shear reinforcement ratio within a wall panel;
$p_{w'}$:shear reinforcement ratio of external shear reinforcement;
p_w .shear reinforcement ratio of a beam or column ($A_v/(b \cdot s)$);
p_{wt} :required shear reinforcement ratio at the middle part of a member;
s :space of shear reinforcement;
t_w .thickness of a wall panel;
α :load concentration factor at the roof floor;
β :ratio of shear carried by the truss mechanism in the effective compressive strength of concrete;
β_{chi} :ratio of shear carried by columns in the i-th story under a higher mode distribution of earthquake forces;
β_{ci} :ratio of shear carried by columns in the i-th story under the fundamental mode distribution of earthquake forces;
β_{whi} :ratio of shear carried by walls in the i-th story under a higher mode distribution of earthquake forces;
β_{wi} :ratio of shear carried by walls in the i-th story under the fundamental distribution of earthquake forces;
γ :unit weight of concrete;
$\Delta\sigma$:stress difference of longitudinal reinforcing bars at the ends of a member;
$\Delta\omega_i$:higher mode coefficient;
Δl_{wa} :incremental length of an equivalent wall width by boundary columns in the arch mechanism;
Δl_{wb} :incremental length of an equivalent wall width by boundary columns in the truss mechanism;
δ_s :projected width of a structural gap,
θ :angle of the compressive strut in the arch mechanism;
θ_{wx} :angle of inclined reinforcement to the axis of member in a wall panel;
θ_x :angle of inclined reinforcement to the axis of member,
κ :coefficient of shear strength of a beam-column connection;
υ :effective factor for the compressive strength of concrete;
υ_0 .effective factor for the compressive strength of concrete in a non- hinged member;

$\Sigma\psi$:total perimeter of longitudinal reinforcing bars,
σ_B :compressive concrete strength,
σ_{sy} :strength of shear reinforcement in a wall panel to estimate the reliable strength;
σ_{wxy} :strength of inclined shear reinforcement in a wall panel to estimate the reliable strength;
σ_{wy} :strength of shear reinforcement in a column or beam to estimate the reliable strength;
σ_{wyu} :strength of vertical shear reinforcement in a wall panel to estimate the upper bound strength;
σ_{xy} :strength of inclined shear reinforcement in a column or beam to estimate the reliable strength;
σ_y :nominal yield strength of a reinforcing bar;
σ_{yu} :strength of longitudinal reinforcing bars to estimate the upper bound strength,
τ_{bu} :bond strength of longitudinal reinforcing bars;
τ_f :bond stress of longitudinal reinforcing bars generated by the flexural moment at both ends;
τ_t :bond stress of longitudinal reinforcing bars generated by shear carried by the truss mechanism;
ϕ :angle of the compressive strut in the truss mechanism,
ϕ_0 :structural strength magnification factor in the yield mechanism assuring design,
ψ_2 :concurrency safety factor,
ω_{ci} :dynamic magnification factor of a column at the i-th story; and
ω_{wi} :dynamic magnification factor of a wall at the i-th story.

[Commentary]

Representative symbols used in this guidelines are annotated in this section. Symbols not listed in this section can be used in the following chapters, whose definition will be given in the corresponding commentaries

CHAPTER 2 . MATERIALS AND MATERIAL STRENGTHS

2.1 Concrete

2.1.1 Materials, Quality and Types

Materials for concrete and their quality shall conform to "Japanese Architectural Standard Specification for Reinforced Concrete Work (JASS-5)" published by Architectural Institute of Japan. Types of concrete by aggregates shall be the normal weight concrete with the design nominal strength (Fc) between 210 kgf/cm^2 and 360 kgf/cm^2.

2.1.2 Material Strength

The compressive strength (σ_B) of concrete can be determined by the design nominal strength (Fc).

[Commentary]

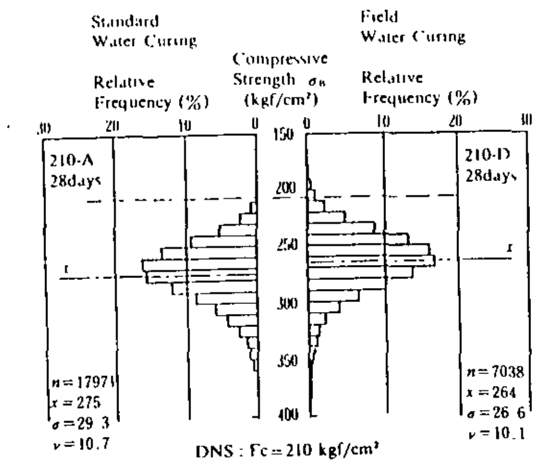
2.1.1 Materials, Quality and Types

The concrete strength used in the evaluation of member strength in an ultimate strength design affects directly the flexural strength of columns and structural walls, and the shear strength of beams, columns, structural walls and beam-column connections. The reliability of strength evaluation methods has been examined for structural member tests using a range of material strengths outside the range defined in the guidelines. However, the range of the concrete strength is determined in accordance with JASS-5 [Ref. 2.1] The use of light weight aggregate concrete is excluded because the reliability of the strength and ductility evaluation has not been established due to lack of test data.

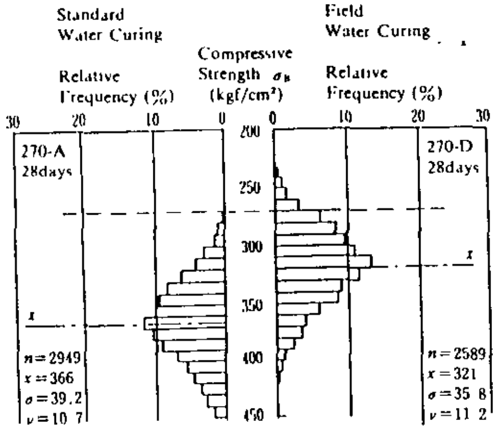
2.1.2 Material Strengths

Reliability of the method to evaluate reliable strengths of a structural member is examined on the basis of actual concrete strength obtained in the test. Therefore, the concrete strength in a design must be the one that could be developed in the construction The variation in concrete strength is described in Ref 2.2. To understand the quality of concrete in recent construction, the relationship between design nominal strengths and standard cylinder strengths, and the relationship between concrete core strengths in various members and design nominal strengths are discussed.

According to the report [Ref. 2.3] from the Japan General Building Research Corporation (GBRC) on the concrete cylinder tests on 24,000 specimens, the average of 28-day strength of specimens by field water curing varies from 1.08 to 1.40 times as large as the design nominal strength with standard variance of 0.097 to 0.136 The risk probability of strength of specimens by field water curing falling below the design nominal strength is approximately 0.5 percent Examples of frequency distribution of the 28-day strength are shown in Fig. C2.1 for different design nominal strengths Coupon specimens by standard water curing show higher strengths than those by field water curing



DNS : Fc=210 kgf/cm²



DNS Fc=270 kgf/cm²

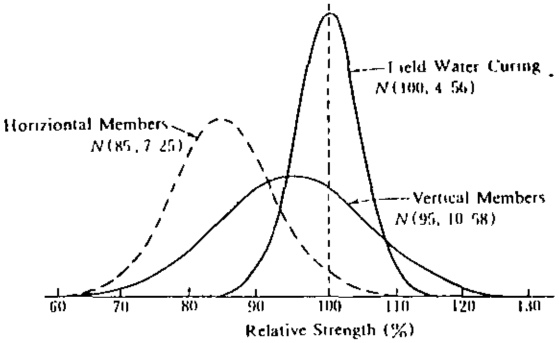
n : No. of Data, x : Average (kgf / cm²), σ : Standard Deviation (kgf / cm²),
v : Coeff. of Variation, DNS : Design Nominal Strength

Fig. C2.1 Distribution of concrete compressive strength.

There are several recent reports on actual concrete strengths in different members [Refs. 2.4 to 2.8]. The work by Takahashi et al. [Ref. 2.8] is quoted herein. The design nominal strength of concrete is 210 kgf/cm². The results of compression tests are summarized in Fig. C2.2 for the concrete taken from full-scale structural specimens (columns, beams, walls and slabs) constructed in usual manner on site.

The 28-day strengths of concrete in both vertical and horizontal members are lower than the 28-day strength of coupon specimens cured by field water curing. However, the 28-day strength of field water curing has the margin of about 1.2 times as large as the design nominal strength, according to Fig. C2.1.

Taking into account the present state of concrete data and 1986 revision of JASS-5, the concrete strength used in calculating the reliable strength is decided to be the design nominal strength Fc. The guidelines assumes the use of concrete in compliance with JASS-5. It is warned, however, that in some cases it has been reported that the concrete strengths in members are less than the specified value due to quality control on the site [Ref. 2.9].



[Note] Figures in N ( ) denote the average and standard deviation, respectively.

Fig. C2.2 Concrete strength in vertical and horizontal members, and coupon specimens by field water curing (28 days).

2.2 Reinforcing Steel Bars

2.2.1 Grades and Sizes

Grades of reinforcing bars shall be SD295A, SD295B, SD345, or SD390, which conform to Japan Industrial Standard (JIS) G 3112 "Steel Bars for Reinforced Concrete." Bar sizes shall be not greater than D38. Deformed PC bars, conforming to JIS G 3109 "Steel Bars for Prestressed Concrete," may be used as shear reinforcement.

2.2.2 Material Strengths

Material strengths for reinforcing bars shall be the value listed in the colum (a) of Table 2.1 for reliable strength calculation, and that listed in the colum (b) of Table 2.1 for upper bound strength calculation

TABLE 2.1 MATERIAL STRENGTHS OF STEEL BARS FOR STRENGTH CALCULATION

Grade	Strength Calculation	
	(a) Reliable Strength	(b) Upper Bound Strength
SD295A, SD295B	1 0 σ	1.3 σ_y
SD345, SD390	1 0 σ_y	1 25σ_y
Deformed PC Bars	1.0 σ_y	————

[Note] σ_y . specified minimum yield strength

[Commentary]

2 2.1 Grades and Sizes

In ultimate strength type design method, the ductility of reinforced concrete members is important; hence, the steel with a good ductility and with known mechanical properties should be used The guidelines limit the use of steel in conformance with JIS G3112 (Steel Bars for Reinforced Concrete), and further of Grades SD295A (Deformed bars of nominal yield stress of 30 kgf/mm^2: 295MPa), SD295B, SD345 and SD390 for both longitudinal and shear reinforcement However, deformed PC Bars conforming to JIS G3109 may be used as shear reinforcement.

If the longitudinal bar is narrow, the bar may buckle after crushing of concrete, lateral reinforcement must be placed at short spacing to prevent buckling On the other hand, if the longitudinal reinforcement is of large diameter, the bond along the reinforcement tends to deteriorate or splitting cracks tend to develop along the reinforcement, and the reinforcement becomes difficult to splice by pressurized gas welding method. Furthermore, the experimental data using the large-diameter bars are also limited. Therefore, the bar size is limited to that smaller than D38 of a nominal diameter of 38mm.

2.2.2 Material Strengths

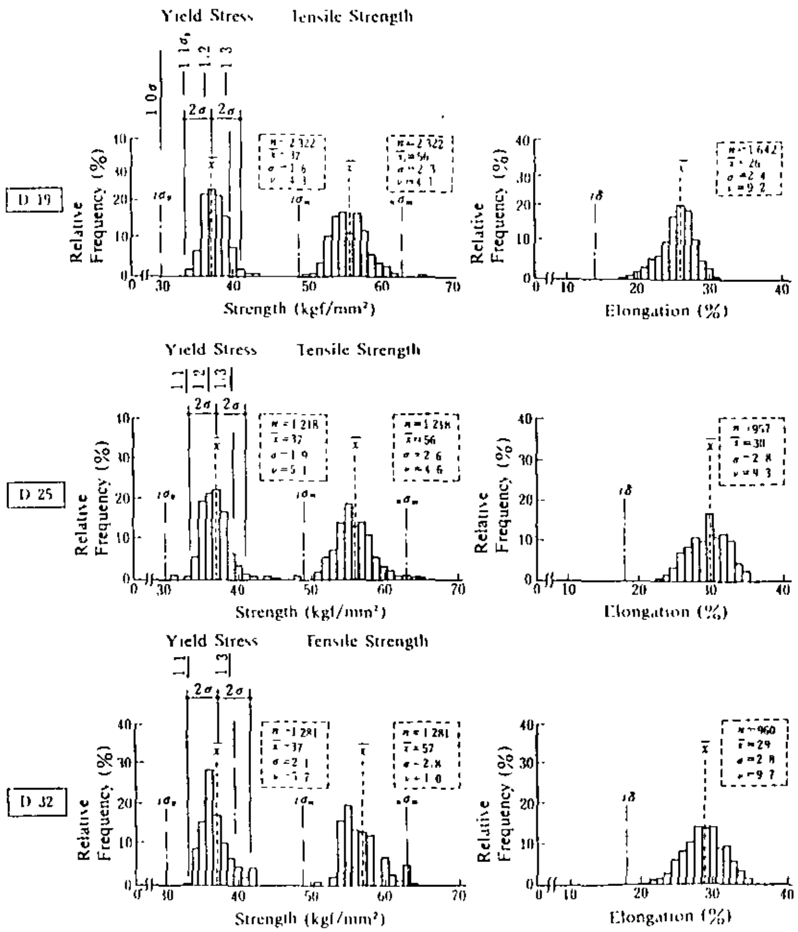
AIJ publication [Ref. 2.2] shows the quality of steel by blast furnace steel producer, especially distributions of yield stress, tensile strength and elongation of deformed bars for reinforced concrete At present in Japan, more than 90 percent of reinforcing steel for building structures in conformance with JIS, however, is produced by electric furnaces. The quality of steel varies with producers, monthly products, bar sizes, and steel grades. Representative data are shown in Figs. C2.3 to C2 5.

According to a report [Ref. 2.10] by Japan Building General Research Institute on the tests of reinforcing bars in tension in 1985, the average yield stress of all bar sizes (from D10 to D38) for grades SD295 and SD345 steel are shown below :

SD295 : average = 1.245 σ_y (standard deviation = 0.07 σ_y)
SD345 : average = 1.142 σ_y (standard deviation = 0.06 σ_y)

The distributions of yield stresses, tensile strengths and elongation are shown in Figs. C2.3 and C2 4 for SD295 (approximately 7,200 specimens) and SD345 (approximately 1,200 specimens) and for different bar sizes (from D19 to D32) [Ref 2 10]

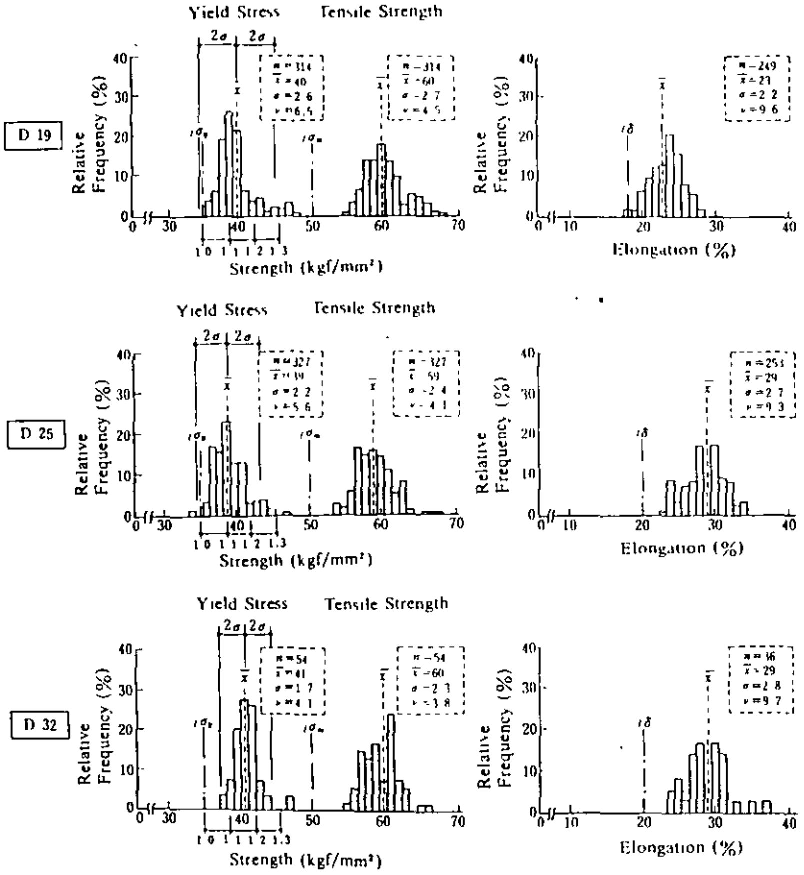
The lower limit (= average - 2 0 x standard deviation) of yield stresses for grade SD295 is approximately 1 1 times as large as the nominal yield strength σ_y for bar sizes from D19 to D32, but the yield stress for use in reliable strength calculation is taken equal to σ_y. The yield stress for upper



[Note] . n : No of Data, $\bar{x}$ Average (kgf / mm^2), σ : Standard Deviation (kgf / mm^2), v Coeff. of Variation (%), $_l\sigma_y$ Min Yield Stress, $_l\sigma_m$ Min. Tensile Strength, $_u\sigma_m$: Max Tensile Strength, $_l\delta$. Min Elongation

Fig. C2.3 Strengths and elongation distribution of SD295 bars.

bound strength calculation is taken to be 1 3 σ_y The average stresses of tensile strength for D29 to D38 bars vary from 5.6 to 5 7 tonf/cm² and appear stable, but the distributions indicate a wide scatter Elongations satisfy the standard. The lower limit of yield stresses for grade SD345 is comparable to nominal yield strength except for D29 and D32 bars The number of data becomes small for grade SD40 steel, the general trend is similar to that of grade SD345 steel. Therefore, the yield stress for reliable strength calculation is taken as σ_y The yield stress for upper bound strength calculation was taken to be 1.25 σ_y although the distribution varies for bar sizes Incidentally, the reinforcing bars produced by electric furnaces occupies 70 percent of the data above

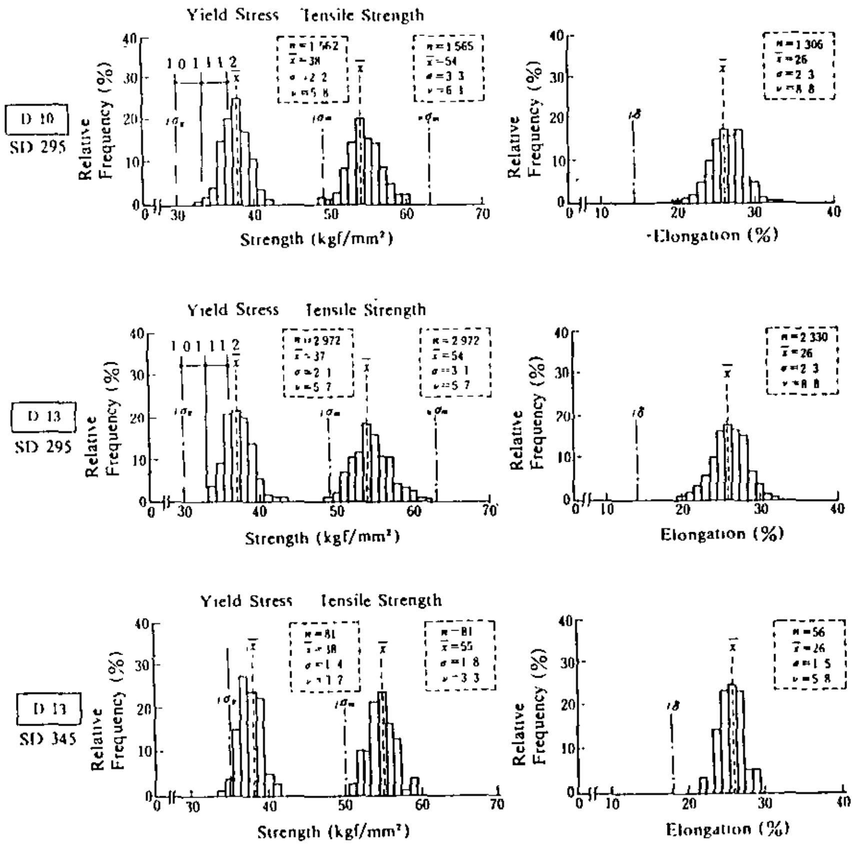


[Note] : n : No. of Data, $\bar{x}$ · Average (kgf / mm²), σ · Standard Deviation (kgf / mm²), v . Coeff. of Variation (%), ${}_l\sigma_y$ · Min. Yield Stress, ${}_l\sigma_m$: Min Tensile Strength, ${}_u\sigma_m$. Max. Tensile Strength, ${}_l\delta$: Min. Elongation

Fig. C2.4 Strengths and elongation distribution of SD345 bars.

. AIJ Standard for Structural Calculation of Reinforced Concrete Structures (1988 Edition) requires the allowable tensile stress of shear reinforcement to be not more than 3,000 kgf/cm² for all steel grade because the number of test data using high strength shear reinforcement has been small Recently, the use of high strength shear reinforcement was developed with an increase in test data. Therefore, the guidelines do not limit the yield stress of shear reinforcement to 3,000 kgf/cm², and allow the use of deformed prestressing steel bars.

Distribution of yield stresses, tensile strength and elongation is shown in Fig. C2.5 for D10 to D13 bars [Ref. 2 10]. From the figure, yield stress for the calculation of reliable shear strength was

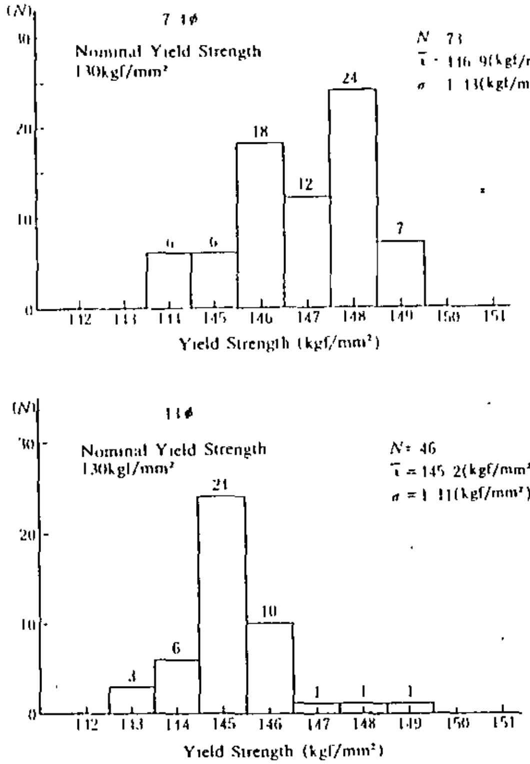


[Note] : n · No of Data, $\bar{x}$: Average (kgf / mm²), σ : Standard Deviation (kgf / mm²), v Coeff of Variation (%), ${}_l\sigma_y$ Min. Yield Stress, ${}_l\sigma_m$: Min. Tensile Strength, ${}_u\sigma_m$ Max Tensile Strength, ${}_l\delta$ · Min. Elongation

Fig. C2.5 Strengths and elongation distribution of small size bars.

taken to be σ_y The use of grade SD390 steel for shear reinforcement is expected in the future The yield stress for reliable and upper bound strengths should be carefully determined on the basis of actual data.

High strength shear reinforcement is currently produced by four producers. The quality is stable. The characteristics of the high strength shear reinforcement produced by Company A are summarized in Table C2 1, and their distributions in Fig. C2 6 Some scatter can be observed in yield stresses, but they never fall below nominal yield strength. Therefore, yield stress for reliable shear strength calculation is taken to be σ_y



[Note] · n · No of Data, $\bar{x}$ · Average (kgf/mm²), σ : Standard Deviation (kgf/mm²)

Fig. C2.6 Strength distribution of deformed PC bars.

TABLE C2.1 CHARACTERISTICS OF HIGH STRENGTH SHEAR REINFORCEMENT

Diameter of Bar		7.4mm	9.2mm	11mm	13mm
	No. of Coupons	73	57	41	46
Tensile Strength (kgf/mm²)	Specified	145	145	145	145
	Average	151	150	150	150.
	Stand Dev.	0 97	0.75	0 94	0.85
Yield Stress (kgf/mm²)	Specified	130	130	130	130
	Average	147	145	146	145
	Stand Dev.	1.43	1 21	1.74	1.11
Elongation (percentile)	Specified	5	5	5	5
	Average	9.4	9.5	10.0	9.6
	Stand Dev.	0.55	0.50	0.42	0.49

2 3 Constants of Materials

Constants of steel and concrete are as listed in Table 2 2

TABLE 2.2 CONSTANTS OF STEEL BARS AND CONCRETE

Material	Young's Modulus (kgf/cm²)	Poisson's Ratio
Steel	2.1×10^6	———
Concrete	$2\ 1\times10^5(\gamma/2.3)^{1\ 5}\ (Fc/200)^{0\ 5}$	1/6

[Note] γ : Weight per unit volume (ton/m³), and may be taken as 2.3 unless values are obtained by a more comprehensive study.

[Commentary]

The constants for materials are taken from AIJ RC Standard.

Young's modulus for the evaluation of stiffness of concrete structures is suggested to refer to Ref 2.2, in which the definition of Young's modulus, correlation of Young's modulus with stress level, influencing factors and examples of actual measurements are discussed.

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CHAPTER 3 : STRUCTURAL PLANNING

3.1 General Principles

3.1.1 Planning of Yield Mechanism

Yield mechanism shall be planned a clean total yield mechanism.

3.1.2 Planning of Members

Locations where yield hinges are intended to develop shall be provided with both necessary resistance and sufficient ductility.

Locations where yield hinges are not intended to develop shall be provided with sufficient resistance.

3.1.3 Planning of Structural Layout and Elevation

Stiffness and resistance shall be uniformly distributed within a horizontal layout planning not to excite an excessive torsional vibration during an earthquake. Stiffness and resistance shall be uniformly distributed along the structure's height as well not to concentrate lateral deflection in a specific story.

[Commentary]

3.1.1 Planning of Yield Mechanism

(1) Earthquake responses of a structure are significantly controlled by the strength (i.e., the lateral load resistance) and energy dissipation capacity of the structure. The fundamental concept of this AIJ Guidelines to achieve seismic resistance and safety of a reinforced concrete building is that to prevent such a large deflection generated as either to endanger serviceability or to lead to total-and/or partial collapse. With this objective, the guidelines require a minimum level of resistance in the building, and to dissipate a significant amount of the vibration energy in a structure making the response deflection less, a clear yield mechanism of the total collapse type should be planned to form in the structure with a great number of yield hinges that are capable of dissipating a large amount of hysteretic energy, distributed within the structure [Refs 3.1, 3 2]

(2) The required minimum level of lateral load resistance of a structure necessary for seismic safety shall be determined for a yield mechanism planned and admissible deflection amplitudes intended To achieve the seismic resisting concept outlined in the guidelines on the basis of ductile performance of an overall structure, the structure shall be carefully designed to form the clear yield mechanism planned, not to generating all other unexpected yield mechanisms during a strong earthquake.

The minimum lateral load resistance shall have been determined on the premise that the planned yield mechanism is developed at the deflection level of the structure that is called the design limit deflection, anticipated in a design earthquake motion, and that the structure keeps the resistance without deterioration beyond the design limit deflection.

In upper stories of medium- to high-rise buildings, where the design for gravity loads determines the proportioning of a member, the intended yield hinges can not develop at the design limit deformation In this case, the required level of lateral load resistance shall be determined not counting the energy dissipation at those locations where hinges are not generated with deformation less than the design limit deformation

3.1 2 Planning of Members

The building shall be provided the required lateral load resistance at the specified design limit deflection, the corresponding required flexural strength should be provided at the planned yield hinge locations The building shall keep the lateral load resistance without deterioration to an assuring deflection beyond the design limit deflection, a sufficient deformation capability should be provided as well at the planned yield hinge locations

The locations of a member where a yield hinge is not planned in design shall be provided with sufficient strength so that the planned yield mechanism is ensured

3 1.3 Planning of Structural Layout and Elevation

(1) A building under design, in principle, shall be planned to have a uniform distribution of mass and stiffness both in the horizontal layout plan and along the height so that a uniform response is developed during an earthquake An unbalanced distribution of mass and stiffness likely leads to a magnification of response in the specific locations of the building, resulting in formation of partial yield mechanisms with concentrated damages in the limited locations

Since it is not desirable to concentrate plastic deflections at specific locations developing yield hinges at very limited localities in a structure, which would result in a partial yield mechanism, the distribution of mass and stiffness both in the horizontal layout plan and along the height of the building shall be planned uniform.

(2) The uniformity in stiffness and mass distribution in the horizontal layout plan and along the height of a building is tentatively evaluated by the so-called eccentricity ratio R_e and story stiffness ratio R_s defined in the Building Standard Law Enforcing Order in Japan [Ref 3.3]. Within the guidelines, provided that the calculated eccentricity ratio R_e is less than the specified figure 0.15, and the story stiffness ratio R_s greater than 0 60 in each story level, the building can be determined regular in shape. The buildings that are determined not regular in shape are excluded from the guidelines.

(3) The eccentricity of strength (resistance) can generate a torsional vibration in seismic response as well as the eccentricity of mass and stiffness [Ref. 3 4] The guidelines could not define the eccentricity of resistance (strength). It is advisable, however, to pay much attention on a uniform distribution of resistance in a building.

[Note· The eccentricity ratio R_e is defined by the ratio of an eccentricity distance to an elastic radius of gyration in each story level; the eccentricity distance is obtained by a distance between the centers of rigidity and mass; the elastic radius of gyration is a square root of the second moment of element stiffness about the center of stiffness divided by the sum of element stiffness The story stiffness ratio R_s is defined by the ratio of r_{si}, which is the reciprocal of the inter-story deflection angle Ri at the i-th story, to that of the average of r_{si}'s taken across all stories; the inter-story deflection angles Ri's are determined under the actions of the fundamental phase design seismic shear force.]

3 2 Frame Building

3.2.1 Yield Mechanism and Locations of Yield Hinges

Yield mechanism, as a general rule, shall be of the beam-yielding type, in which yield hinges develop at the ends of all floor beams and at the base of the first story columns

3.2 2 Exceptions in Locations of Yield Hinges

In planning of the yield mechanism, yield hinges may form at the following locations;

(1) at the top of the top story columns,

(2) in exterior columns where axial loads are reduced by seismic lateral forces,

(3) in interior columns that are not intended to share seismic lateral loads in the design.

[Commentary]

3.2.1 Yield Mechanism and Locations of Yield Hinges

(1) It is desirable to plan the structure to form a beam-yielding mechanism (Fig. C3.2) to dissipate a vibration energy generated by an earthquake excitation through large ductility and inelastic hysteresis of a structure

The reasons to plan yield hinges at beam ends forming a beam-yielding mechanism are:

a) A beam is easy to ensure a large flexural ductility since no large axial load works.

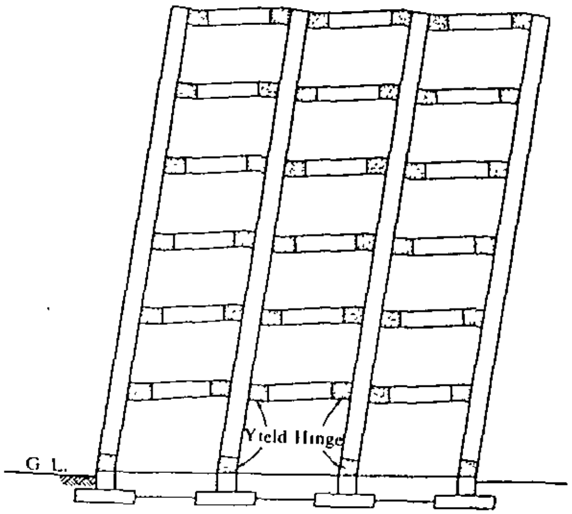

Fig. C3.2 Total yield mechanism of beam-yield type.

b) A hysteresis loop by yielding of a beam is stable, and can dissipate a significant amount of energy.

c) Damage by yielding at beam ends can be repaired relatively with ease, and resistance restored.

The reasons to plan the beam-yielding yield mechanism are:

a) Beam ends are located large in number throughout a building, and they can be planned to develop yielding hinges simultaneously to form a total yield mechanism. Whereas a partial yield mechanism forming yield hinges at a limited number of locations can develop a large deflection, a beam-yielding total mechanism can develop a moderate deflection, dissipating a large amount of hysteretic energy at yield hinges located at beam ends scattered throughout the building.

b) The flexural yielding at beam ends resulting from formation of the beam-yielding mechanism will not directly lead to collapse of the building

(2) Since columns are normally subjected to large axial loads, they are difficult to develop a large plastic deformation capacity. Through an experimental study on a column subjected to lateral and varying axial loads combined that corresponds to the lateral deflection simulating an exterior column of a frame building [Ref. 3.9], under tensile loading, the column resistance is found not so large, while the column develops a large capacity On the other side under increasing compressive loading, the resistance decays rapidly beyond the maximum resistance is obtained, and the deformation capacity is found limited under compression.

Columns sustain the weight of a building. Significant damage in columns, therefore, will directly lead to collapse of a building

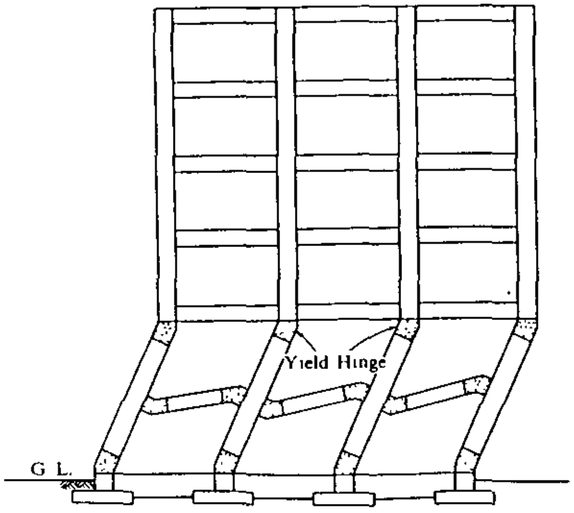

Fig. C 3.4 **Partial yield mechanism by forming hinges in columns.**

Furthermore, yielding at column ends can lead to a story yield mechanism that will form a partial yield mechanism as shown in Fig C3.4, in which the vibration energy shall be dissipated at the limited yielding stories, resulting in a significant plastic deformation in the yield hinges and deflection at the yielding stories that develop the so-called beam sidesway mechanism.

(3) Even to intend to form a beam-yielding total mechanism, yield hinges should be developed at the bottom of the first story columns Foundation girders, connecting the base of the first story columns, are normally provided with high stiffness and strength to avoid uneven settlement of the foundation; therefore, the foundation girders are difficult to develop yield hinges. Furthermore, it is not advisable to form yield hinges in the foundation girders by the following reasons:

(a) A foundation girder, normally located under the ground, is difficult to examine damage after an earthquake, and it is difficult to repair the damage

(b) The yield hinges of foundation girders are not desirable for the stability of the building

From the reasons mentioned above, the beam-yielding total mechanism shall be planned with yielding at the base of the first story columns

3.2.2 Exceptions in Locations of Yield Hinges

(1) A yield hinge can be allowed to form at the top of top story columns because the column is normally connected to two beams at the joint. Beams at the upper story levels are on frequent occasions determined from the design for gravity loads Therefore, unless the formation of a yield hinge is not allowed at the location, the design actions in the column would become excessive.

The following reasons can be pointed out to allow a yield hinge at the top of a top story column:

a) The axial load in the column is small, therefore, a large deflection capacity can be easily expected at the location.

b) Hysteretic energy, comparable to that at the beam ends, can be expected dissipated at the location.

c) A yielding in a top story beam may lead to the damage of parapet in water proofing of the roof that will require a serious repair work

(2) An exterior column, i.e., a column positioned at the exterior of the frame, is subjected to a large amount of tension and compression due to an overturning produced by lateral loads When the axial force in a column gets tensile, the flexural resistance is reduced, causing flexural yielding in the column.

An empirical study has indicated an evidence that a column under tensile axial force reveals a significant deformation capacity.

A major reason for avoiding column yielding is a difficulty to ensure a large deformation capacity, since a column generally carries a high axial compressive load. A column subjected to a tensile load or less high compressive load, however, can develop a large ductility

(3) An interior column, i e , a column positioned within a frame, that is intended to share no seismic lateral load, and the design of which is governed by actions from the gravity loading, can be allowed to form a yielding Yielding of such a column is not likely to lead to the formation of a partial story yield mechanism The column, however, should sustain the gravity load

3.3 Structural Wall-frame Building

3.3.1 Layout and Configuration of Structural Walls

Structural walls, as a general rule, shall be positioned in symmetric locations to form a regular structural plan, and shall be continuous from the bottom to top stories

3.3.2 Openings in Structural Walls

If an opening is placed in a structural wall, the effect of the opening on stiffness and resistance of the wall shall be taken into consideration.

3.3.3 Locations of Yield Hinges in Structural Walls

A structural wall shall be planned to form a mechanism yielding in flexure at its base A mechanism rotating at its base resulting from uplifting of the foundations prior to yielding in flexure at the base can be intended as well

3.3.4 Yield Mechanism of Frames

Yield mechanism of frames within a wall-frame building, as a general rule, shall be of the beam-yielding type in each. However, if structural walls are provided a sufficient amount of resistance, yield hinges can be developed in columns

[Commentary]

3.3.1 Layout and Configuration of Structural Walls

A structural wall-frame structure, in a general form, is constituent of structural walls coupled with a set of beam-column frames.

It is desirable that structural walls are arranged in symmetry in the horizontal layout to ensure a regular stiffness distribution in plan, and are placed in continuity from the base to top of the building not to produce an abrupt discontinuity in both stiffness and strength along the height of the building Irregular configuration of a structural wall will cause unfavorable yielding that can result in forming a partial yield mechanism not intended in design.

A structural wall that is located within smaller number of spans in lower stories is completely excluded in structural planning. A structural wall with a setback configuration, which has smaller number of spans in upper stories, is, in a general rule, not intended in the design since of the undesirable discontinuity of stiffness and resistance of the building.

In the basement, rigid and strong walls are placed along the periphery. In this case, a yield mechanism of a structural wall is often formed clearly by a flexural yielding at the base of the first story Therefore, the base of a structural wall can be either widened in span and enlarged in thickness in the basement.

3.3.2 Openings in Structural Walls

If an opening is placed in a structural wall for such penetration for service facilities, stiffness and strength of the wall should be reduced according to shape and location of the opening, and the periphery of the opening should be reinforced A method to reduce stiffness and strength by an opening is not in particular introduced in the guidelines When employing a conventional method for reinforcing the periphery of the opening such as that proposed in the AIJ Calculation Standard, one should pay attention on the planning that a structural wall designed in conformance with the guidelines is expected to develop a larger deformation capacity after yielding than that required to other structural walls usually intended non-ductile

3.3.3 Locations of Yield Hinges in Structural Walls

(1) The planned yield mechanism of a structural wall, in principle, shall be the one developing a flexural yield hinge at the base of the wall as shown in Fig C3.6 If the stiffness of a structure is dominantly provided with the structural walls, the deflection of the building is determined by that of the structural walls; the deflection is controlled resulting in an almost uniform shape along the height of the building [Refs. 3.11 and 3.12].

As illustrated in Fig C3.6, flexural yield hinges are essentially formed at beam ends on the boundary with the structural wall unless other failures such as a shear yielding are formed in the beams, and at beam ends on the other side unless the columns are provided no greater strength than necessary

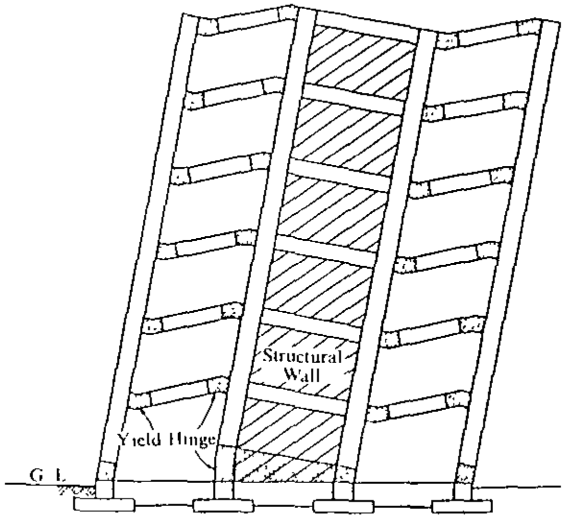

Fig. C 3.6 Total yield mechanism for a structural wall-frame building forming a flexural yield hinge at the base of a structural wall.

(2) Unless a basement is planned, a yield mechanism resulting from rotation of the wall caused by an uplifting of foundation slabs can take place as shown in Fig C3.7. Yield hinges will develop in girders including foundation girders that are connected to the wall, which will results in forming a beam-yield type mechanism In this case, difficulty is expected to examine damage in the foundation, and to repair the damage after an earthquake

Both yield mechanisms of flexural yield hinges of the wall and rotation of the wall foundation are ductile. A yield mechanism of shear failure of the wall, however, is brittle.

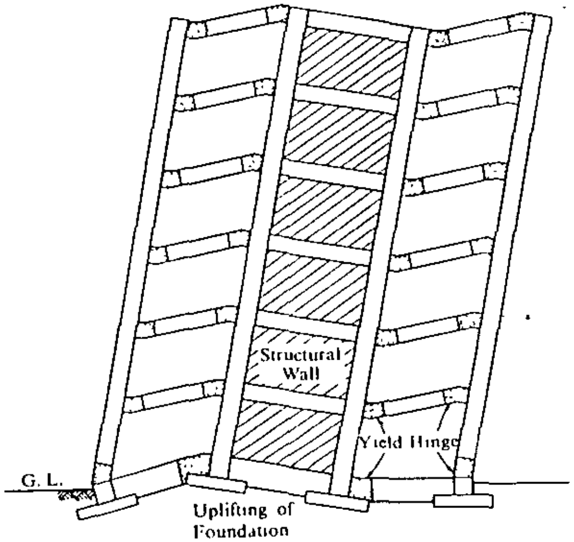

Fig. C 3.7 Total yield mechanism for a structural wall-frame building developing rotation of the wall caused by uplifting of the foundation.

(3) If a basement is intended in a building, a structural wall shall be planned to develop a flexural yielding at the base of the first story. Since the stiffness and strength of basement walls are greater than structural walls, it will be found most probable that yield hinges shall be formed at the base of the first story level provided that other failures such as a shear failure of the wall are countered. Structural performance is clear with the yield hinges in flexure at the first story base.

When the one structural wall forms a flexural yield hinge at the first story level and the other will form a flexural yield hinge at the bottom of the wall (at the basement level) or form an uplifting rotation of the foundation, compatibility condition in deflection between the walls will generate deformation in slab members connecting the walls, or deformation in the walls not initially intended in design. Unless yield hinges develop at the first story level, it is hard to intend a clear structural performance, and the building can develop such an unexpected yield hinge mechanism that is undesirable.

(4) If a pile foundation is employed under a structural wall, the every resistance against wall uplifting should be considered including the counter-moment contribution of parallel and orthogonal foundation girders, weight of foundation and piles, and others At the present, due to lack of both empirical and analytical results, it is difficult to evaluate those resistances such as the pull-out resistance of piles. If the pull-out resistance of a pile were underestimated in the uplifting mechanism, the structural wall can have higher resistance against the foundation uplifting than expected, which can develop a shear failure of the wall that is not intended in design

When resistance of the wall can be determined by such as the pull-out resistance of a pile, one shall intend to provide the wall with sufficient shear strength not to generate a brittle shear failure taking a significant excess of resistance into consideration.

3 3.4 Yield Mechanism of Frames

In a structural wall-frame building, the total yield mechanism can be obtained by preventing to form a story mechanism of the structural wall as indicated in Figs C3.6 and C3.7. If a structural wall is provided with a sufficient strength to prevent a shear failure, frames within a structural wall-frame building will not necessarily develop the beam-yield type mechanism in an individual frame to produce an overall deflection uniformly distributed within a building. If structural walls govern the stiffness of the building, the overall deflection is determined by that of the structural walls. Consequently, formation of yield hinges in columns will not always produce an excessive amount of deflection at specific story levels that will lead to a partial story mechanism.

When strengths of columns are taken smaller than those of beams in frames, an equilibrium condition suggests formation of column-yield type mechanism. While the formation of yield hinges in columns does not lead to a significant deflection at specific story levels resulting in a story mechanism, it is not desirable and advisable from a point view of dissipating a large amount of hysteretic energy and ensuring great deformation capacity in yielding members planned in design. Based on the reasons mentioned above, it is desirable that a beam-yield type mechanism is ensured for each frame even in a structural wall-frame building.

3.4 Foundation and Basement

3.4.1 Foundation Girders

Yield hinges, as a general rule, shall not be planned at ends of foundation girders However, when a structural wall is positioned to form a yield mechanism by uplifting at the foundation, yield hinges can be planned at foundation girder ends.

3.4.2 Foundation Slabs and Piles

Yield hinges, as a general rule, shall not be planned in either foundation slabs or piles

3.4.3 Basement

Basement of a structure shall be provided with sufficient stiffness and, as a general rule, shall not be planned to form yield hinges

[Commentary]

3.4.1 Foundation Girders

Foundation girders should not, as a general rule, form a part of the planned yield mechanism. The planning of yield hinges at the foundation girders could be advisable from the point of view that yield hinges are not generated at the base end of the first floor columns that are hard to ensure sufficient ductility because of their high axial stresses. From other points of view that yield hinges in a foundation girder, are hard either to be examined and to be repaired after an earthquake, and that foundation including the foundation girders shall provide a structure with stable support, it should be desirable not to form yield hinges in foundation girders.

When a structural wall is positioned within a high- or medium-rise building, an uplifting rotation at its base can be formed, which is one of the allowable yield mechanisms since it can dissipate vibration energy efficiently without affecting the resisting capacity. When the wall is uplifted, it is inevitable to form yield hinges at foundation girders at the wall boundary. In the case to allow yield hinges formed at foundation girders, damage inspection and repair after an earthquake should be taken into consideration in structural planning.

3.4.2 Foundation Slabs and Piles

Both foundation slabs and piles shall not, in principle, form a part of the planned yield mechanism. Similar to foundation girders, it is troublesome in the foundation slabs and piles to examine damage and to repair the damage. As a part of the foundation structure, they are expected to support firmly the structure.

3.4.3 Basement

It is not desirable to form yield hinges in the basement for the reason that it shall settle the structure stable to that described for foundation girders, slabs and piles. The basement is, in general construction, provided with sufficient strength and stiffness.

3.5 Non-structural Members

> **3.5.1 Influence on Structural Members**
>
> Connection between non-structural and structural members shall be planned not to give influence on the planned yield mechanism of a structure.
>
> **3.5.2 Countermeasures against Damage**
>
> Damage and the resultant peeling-off of non-structural members shall not interfere with various building functions such as those necessary for emergency evacuation.

[Commentary]

3.5.1 Influence on Structural Members

Those non-structural members shall be removed that can influence the structural members on the formation of the planned yield mechanism or, in some cases, that can influence the structural members to form a yield mechanism other than initially planned.

The connection of non-structural members to structural members shall be carefully exercised so that non-structural members should not be damaged in a small to moderate earthquake, and that non-structural members should be structurally disconnected from structural members during a severe earthquake when the planned yield mechanism can probably be formed.

3.5.2 Countermeasures against Damage

In such a severe earthquake as to develop a yield mechanism of a structure, (1) non-structural members shall be properly supported by themselves not causing peeling-off that might disturb the evacuation or jeopardize the safety of occupants, and (2) non-structural members shall be properly separated from structural members not to give influence on the structural members in formation of the planned yield mechanism, and shall not generate an excessive damage to themselves due to the separation from structural members.

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CHAPTER 4 . DESIGN METHOD

4.1 Design Principle

4.1.1 Design Objectives

The objectives of structural design shall be to ensure both strength and serviceability necessary for gravity loading and for medium intensity earthquake motions, to assure the formation of a ductile total yield mechanism during a strong earthquake motion and to ensure a minimum resistance required not to develop an excessive deformation.

4.1.2 Design for Gravity Loading

Strength, deformation, uneven settlement, cracking and vibration shall be examined under gravity loading.

4.1.3 Design for Earthquake Loading

Design for earthquake loading is performed both in the yield mechanism design and in the yield mechanism assuring design.

(1) In the yield mechanism design, planned yield hinges shall be provided with the reliable strength required for the specified lateral resistance of a structure and with sufficient ductility

(2) In the yield mechanism assuring design, the regions other than the planned yield hinges shall be provided with the reliable strength large enough to prevent failure even under upper bound actions during a strong earthquake motion.

[Commentary]

4.1.1 Design Objectives

Two intensity levels of earthquakes are considered for building design; i.e., moderate and strong earthquakes. Acceptable damage for the two intensity level is prescribed as follows

For moderate earthquakes: For earthquakes which may occur several times during a lifetime of the building, damages which can be restored by minor repair works can be accepted; i.e., cracking of concrete but no plastic deformation accompanying steel yielding

For strong earthquakes: For an earthquake which possibly may occur once in a life time of the building, damages which can be restored by substantial repair works can be considered Maximum story drift (Design Limit Deformation) may reach as large as 1/100 drift angle. For the building, deformation capacity (Assurance Deformation) sufficiently larger than the design limit deformation should be provided at the planned yield hinges.

The intensity of an earthquake motion is influenced by the seismic mechanism, wave propagation path and local soil conditions. The earthquake resistant design should consider an earthquake that may have the most destructive effect on buildings Within the design guidelines the intensity of design ground motion has not been definitely specified The intensities of moderate and strong earthquakes, however, may be indicated in terms of either maximum ground velocity or acceleration for reference

Moderate earthquake motions : 100-120 cm/sec^2 in acceleration
15- 20 cm/sec in velocity
Strong earthquake motions : 300-400 cm/sec^2 in acceleration
40- 50 cm/sec in velocity

Samples of real earthquake motions widely used in a nonlinear earthquake response analysis are listed with the maximum amplitudes

EL Centro (1940) NS Component : 342 cm/sec^2, 33 cm/sec
Hachinohe Harbor (1968) EW Component : 180 cm/sec^2, 38 cm/sec
Tohoku University (1978) NS Component : 258 cm/sec^2, 36 cm/sec

Although the design guidelines considers two levels of earthquake intensity, the design criteria are mainly established for the strong earthquake motion. Member actions developed in the response for a moderate earthquake motion will be assumed smaller than design actions under the design earthquake loads. When the member actions reach the stress level calculated by a linear structural analysis under the design earthquake loads, the total yield mechanism will be formed, in principle, by simultaneous flexural yielding at the planned hinge locations

Since some damages can be tolerable in moderate earthquake motions, the redistribution of the design moments is allowed to optimize strength distribution and consequent reinforcement arrangement in a structure.

If the design moment redistribution will be performed without limitation, the location, where design moment determined from a linear analysis is reduced, is expected to develop yielding from a moderate earthquake motion and/or suffer a significant damage from a strong earthquake motion. Consequently, a limitation is placed on the amount of design moment redistribution.

4.1.2 Design for Gravity Loading

The Steering Committee of Reinforced Concrete Structures, Architectural Institute of Japan (AIJ), has organized a sub-committee on Design for Long-term Loading, in which a design method for long-term loading on the basis on the ultimate strength concept has been under discussion. Since the discussion has not been finalized in the sub-committee, the design for gravity loads may tentatively follow the "AIJ Standard for Structural Calculation of Reinforced Concrete Structures [Ref. 4.1]" based on the allowable stress design format.

Dead load must be calculated from structural and architectural design documents. The values for the live load, snow load, and wind load may be given in the Building Standard Law and associated Provisions or the "AIJ Guideline for Loads on Buildings and Commentary [Ref. 4 2]."

For the gravity load that acts constantly in a building, adequate strength should be ensured necessary for safety, and every part of the structure should be provided with sufficient stiffness to prevent excessive deformation, uneven settlement, vibration of slabs and other issues associated with serviceability or deterioration of durability due to a large cracking.

4.1.3 Design for Earthquake Loading

Horizontal load carrying capacity of a structure is determined from the strength of yield hinges in the planned total yield mechanism In the yield mechanism design (Mechanical Design), the mode of yielding at yield hinges must be selected to be of ductile type, i.e., of flexural yielding type, in principle. The yield hinges must be provided with the reliable flexural strength associated with a required horizontal load carrying capacity, and also provided with ductility to allow sufficient plastic deformation.

In the yield mechanism assuring design (Assurance Design), regions other than the planned yield hinges required for the formation of the total yield mechanism shall be provided with adequate reliable flexural strengths. And furthermore every part of the structure should be provided with resistance against other failure modes than flexure so that the structure is assured to form the planned yield mechanism. For the planned yield hinge regions ductility shall be provided, for other regions, however, ductility is not necessarily required.

To satisfy the above design requirements, the mechanism design and assurance design use their own design member forces determined for the mechanism design and assurance design, respectively.

Member design forces for the mechanism design are determined by a linear analysis under static design earthquake loading, taking deterioration of member stiffness into consideration and allowing moment redistribution with acceptable limitation. Member design forces for the assurance design are determined by a non-linear analysis at the formation of the planned yield mechanism assigning the upper bound flexural strengths at the prescribed yield hinges calculated for the actual amount of reinforcement, considering dynamic amplification effects and bidirectional earthquake response effects into account

The following three deformation levels shall be considered (Fig C4-1).

(1) Yield Deformation

(2) Design Limit Deformation

(3) Assurance Deformation

Yield deformation corresponds to the deformation at the yield point on the load-displacement relation as a total structure, and ideally signify the deformation when all the planned hinges are formed simultaneously under the static design earthquake loading. Since in general cases, however, each yield hinge reaches its yield strength at different deformation level, the yield deformation of a structure is hard to be defined. The design guidelines requires the yield deformation of a structure to be smaller than the specified value so that the stiffness of a structure should be maintained and that the resistance of the structure could be fully developed prior to an inelastic response deformation (design limit deformation) during a strong earthquake motion.

Design Limit Deformation is a limiting deformation by the design earthquake motion. The design guidelines assumes this deformation to be 1/100 radian in terms of the story drift angle.

Assurance Deformation is the upper bound earthquake response deformation considering the deviation of probable uncertainties included in such as intensity level and spectral characteristics of strong earthquake motions, site subsoil conditions, performance of the designed structure and the method employed in an inelastic dynamic analysis The inelastic deformation capacity (ultimate deformation and ductility) at yield hinges must be adequate for the assurance deformation described herein. In design, the assurance deformation shall be selected much larger than the design limit deformation in order to ensure a deformation capacity of the structure

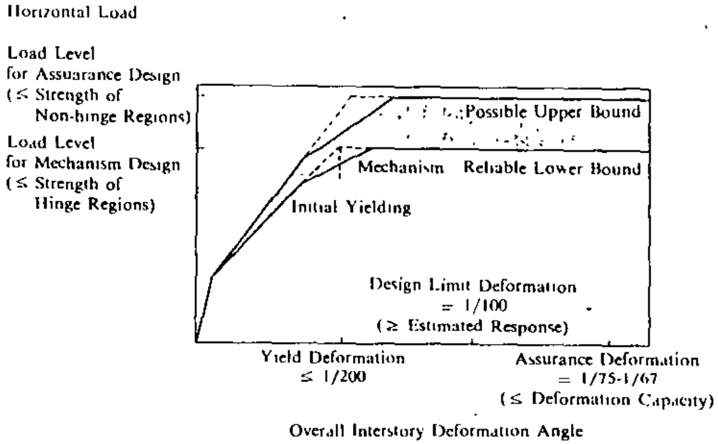

Fig. C4.1 Design loads and deformations.

4.2 Design Loads and Their Combination

The dead load, live load, snow load, wind pressure, earthquake forces shall be combined using appropriate load factors.

[Commentary]

Loads for design of girders, columns and foundations that shall be considered with earthquake loads are dead load, snow load, wind pressure and live loads, which are specified in the Building Standard Law and accompanying Provisions The building weight for the calculation of earthquake loads includes the dead and reduced live loads specified for the calculation of earthquake loads in the Law. All load factors are tentatively specified as unity until new information is provided from research and development.

Ultimate strength design method (load factor-strength reduction factor design method) or limit states design method requires a member strength to be greater than the member force under the most severe combination of loads. For example, ACI-1983 specifies the load combinations as follows

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[Note: A resistance of a structure cannot be fully utilized if the earthquake response deformation is smaller than the deformation at the maximum resistance. To expect the full resistance, providing a structure with adequate stiffness the resistance should be developed within a deformation range generated during an earthquake.]

$$U = 1.4 D + 1 7 L \qquad \text{(Eq. C4.1)}$$

$$U = 0 75 (1 4 D + 1 7 L + 1 1x1 7 E) \qquad \text{(Eq C4.2)}$$

$$U = 0 9 D + 1.1x1 7 E \qquad \text{(Eq. C4.3)}$$

where D, L and E denote the dead, live and earthquake loads respectively.

In Eq. C4.1, the dead load can be estimated accurately, while the live load including snow loads is fluctuated depending on circumstances of the building Therefore, the load factors for dead and live loads are specified in different manners, and the safety of the structure is ensured even for simultaneous development of maximum amplitudes. In Eq. C4.2, it is recognized that the probability of simultaneous occurrence of the maximum dead, live and earthquake loads is low; hence, the combined load is reduced to 3/4. In Eq. C4 3, it is considered that the columns under lower axial loads may develop smaller flexural strength.

This design guidelines specifies the load factor for earthquake load to be unity since the value of horizontal load carrying capacity at the formation of the yield mechanism is specified.

The load factors for gravity loads are unity as well. The flexural strength of columns is affected by the amplitude of axial load. Therefore, columns should be designed under both maximum and minimum axial loads Since the design guidelines assumes the total yield mechanism of beam yielding type, the yielding in columns can occur exclusively at the bottom end of the first story columns. Although flexural strengths of the first story columns may be affected, the effect of variation in the axial load on the total amplitude of horizontal load caring capacity will be small. A large gravity load on the beams may cause the upper reinforcement of the member to yield under a small lateral load, but this action may also delays the yielding of the lower reinforcement of the member. As a result, the horizontal load carrying capacity is almost invariant associated with the changes in gravity loads. This design guidelines assumes an average gravity load at the formation of the yield mechanism under a strong earthquake motion. The examination at maximum or minimum gravity loads may be abridged

Loads by wind pressure, soil pressure, water pressure, vibration and impact forces need not be combined with the earthquake load for simplicity and abbreviation of design procedure.

4.3 Yield Mechanism Design (Mechanism Design)

4.3.1 Design Earthquake Loads

(1) In the yield mechanism design, the design base shear coefficient shall be given by Eq (4.1).

$$C_1 = Z R_t C_B \qquad (4.1)$$

where C_1, Z and R_t denote design base shear coefficient, earthquake zone factor and vibration characteristic factor, respectively, and C_B designates the standard base shear coefficient that shall be not less than 0.25 for a frame structure and 0.30 for a structural wall-frame structure.

(2) The design earthquake load may be assumed to act in the two principal directions separately. The horizontal load at the i-th floor can be given by Eqs. (4.2) to (4.4) unless determined by a special study.

$$\begin{aligned} F_n &= P_t + P_n && \text{for the top floor} \\ F_i &= P_i && \text{for the i-th floor } (i < n) \end{aligned} \qquad (4\ 2)$$

in which P_t is given by Eq (4 3) P_t may be taken as zero for a structural wall-frame structure and for a frame structure of not more than 6 stories

$$P_t = \alpha T Q_1 \qquad (4.3)$$

And P_i is given by Eq (4.4) :

$$P_i = (Q_1 - P_t) W_i H_i / (\Sigma W_j H_j) \qquad (4.4)$$

where F_i, Q_1, α, T, H_i, H_n, W_i and n denote the horizontal load at the (i + 1)-th floor, design base shear, load concentration factor at the top floor to be specified 0.10, fundamental period of the structure (= 0.02 H_n), interstory height, total height of the structure from the ground level in meter, sum of dead and live loads for earthquake loading at the (i + 1)-th floor, and number of stories of the building, respectively.

4.3.2 Linear Analysis

Design actions in the yield mechanism design shall be based on a linear structural analysis using realistic member stiffness and the following assumptions.

(1) For members intended to develop yield hinges, member stiffness shall be a secant stiffness at flexural yielding For members not intended to develop yield hinges, member stiffness shall be appropriately reduced from elastic stiffness reflecting cracking at the calculated stress level

(2) Moment of inertia to determine member stiffness shall be calculated for the gross section For members having T-shape sections such as T-shape beam and columns with orthogonal walls, effective width of flange members shall be taken into consideration. Contribution of reinforcing bars may be neglected.

(3) Shear deformation shall be considered in structural walls.

(4) Stress transfer between adjacent parallel frames through diaphragm shall be considered appropriately.

4.3.3 Moment Redistribution

Member actions in the yield mechanism design obtained by the linear analysis may be redistributed within the following restrictions.

(1) Redistributed moments must satisfy the equilibrium conditions with the design loads.

(2) Redistributed moment shall be not greater than 20 or 25 percent of the moment obtained by the linear analysis under the design earthquake loads in a frame structure or a structural wall-frame structure

(3) Change in sum of the total joint moments at a floor accompanied by the moment redistribution shall be not greater than 5 or 15 percent of the sum of the total joint moments obtained by the linear analysis under design earthquake loads in a frame structure or in a structural wall-frame structure

4.3.4 Deformation Limit

Story drift calculated under the design earthquake loads shall be not greater than 1/200 of the story height.

[Commentary]

4.3.1 Design Earthquake Loads

The required ultimate horizontal load carrying capacity in the second design stage of the current Japanese design procedure is at least 1.5 times as large as that in the first design stage based on an allowable stress method. Especially in design of low-rise buildings, the requirements in the first stage design may be sufficient for the required horizontal load carrying capacity of the second stage design, because additional reinforcement may be placed by the following reasons:

(1) minimum reinforcement requirements,
(2) use of oversized bars and overreinforcement for convenience in construction, and
(3) rounding of required number of reinforcing bars.

Furthermore, in the second stage design, some effects that are ignored at the first design stage, may be considered such as:

(1) material strength is 1.1 times as large as the nominal strength,
(2) ultimate strength design equation is employed to evaluate the member strength;
(3) rigid zone at a beam-column connection is considered;
(4) slab reinforcement for beams and intermediate reinforcement in columns may be taken into account in strength evaluation; and
(5) effect of transverse members is taken into account in strength evaluation

The level of earthquake design load for the yield mechanism design does not correspond to the levels of the either first and second design stages in the Building Standard Law. This design guidelines appears to select the structural characteristic coefficient (Ds), which is described in the Building Standard Law, to be smaller than the current Ds values by 0.05 because the standard base shear coefficient is taken as 0.25 for a frame structure and 0.30 for a wall-frame structure. The coefficients are not comparable because both the proposed design procedure and member strength evaluation method are different from ones in the current design method. For example, the steel strength taken 1.1 times the nominal yield stress is used in the calculation of the horizontal load capacity in the current design practice, while the nominal yield stress is used in the design guidelines. Also in the guidelines, an additional strength due to construction steel on site may not be included because it will not be effective prior the design limit deformation is reached.

The strength levels of the proposed design guidelines and the Building Standard Law cannot be directly compared because both the design method and definition of resistance are different; however, the resistance at yield hinges will almost equal each other for the two design methods, while the resistance in regions other than at the yield hinges from the proposed guidelines will be much greater than that from the Building Standard Law. A probable reason comes from the fact that the earthquake design load level for the yield mechanism design is selected in conformance with the Building Standard Law and its accompanying Provisions. It is desired that the earthquake design load level should be reviewed in near future by examining expected earthquake intensity and their structural response and by reflecting social and economical demands.

Compared to a structure designed in accordance with the Building Standard Law and accompanying Provisions, a structure designed according to the design guidelines will behave well during a strong earthquake motion. Although the resistance at the yield hinges may be lower for the guidelines than for the Building Standard Law, this design guidelines requires the provisions as follow that are not required in the current design provisions

(1) Structural Planning

The design guidelines is not applicable to irregular shape buildings or buildings with less deformation capacity. With the present state of the art, dynamic response of a structure to a strong earthquake motion is hard to predict with sufficient accuracy, and rational design provisions are difficult to establish with confidence. The scope is expected to be widened with development of future researches.

(2) Assurance of Total Yield Mechanism

Regions of members other than at the planned yield hinges are designed to resist amplified design forces to cover uncertainties. A structure designed by the Building Standard Law may develop a story mechanism during a strong earthquake motion. The total yield mechanism, no doubt, requires less concentration of plastic deformation in constituent yield hinges, hence less ductility at the planned yield hinges. Therefore, a structure ensured to develop the total yield mechanism performs good during a strong earthquake motion.

(3) Ductility at Planned Yield Hinges

The planned yield hinges are designed to assure a plastic deformation capacity to the assurance deformation, which is specified greater than the maximum response deformation in an expected strong earthquake motion. Therefore, the yield hinges are believed to maintain their resistance in an earthquake having greater intensity.

(4) Realistic Structural Stiffness in Analysis

Yield deformation under the design earthquake loads is estimated in realistic manners by using reduced stiffness due to such as cracking, and the value of the yield deformation is limited to ensure stiffness of the structure

(5) Design of Beam-column Connection

The design of a beam-column connection not having been required in the Building Standard Law and its accompanying Provisions is clearly introduced in the guidelines

(6) Reinforcement Detailing

Additional reinforcement detailings are required in the design guidelines.

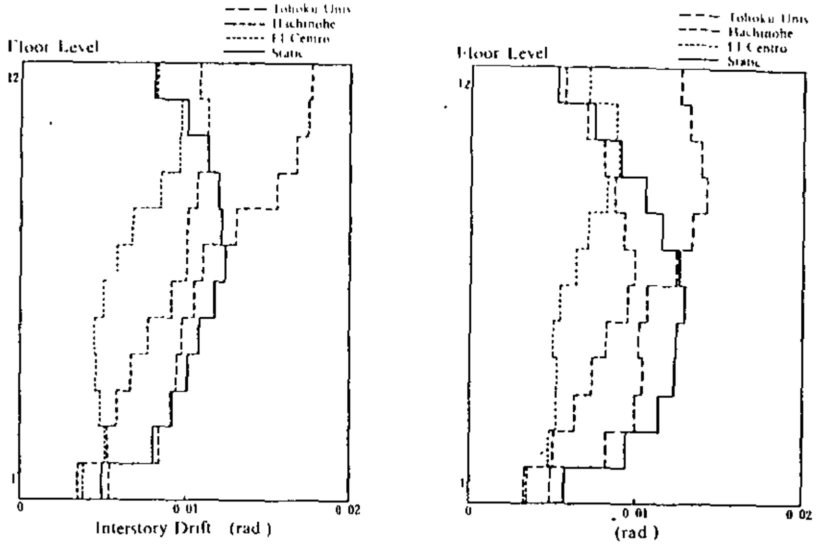
[Note: Elastic earthquake design forces are reduced by the structural characteristic coefficient Ds, counting on ductility of constituent members in the second stage design in the Building Standard Law. For a moment resisting frame consisting of high ductile reinforced concrete members, Ds of 0.30 is taken.]

The earthquake design load distribution along the height of a structure, which will determine the strength distribution at the planned yield hinges, is of an inverted triangular shape that approximates an average distribution of earthquake loads observed in nonlinear earthquake response analyses. For a building which develops the total yield mechanism, the distribution of beam strengths along the height may not control the performance of the structure provided it is given the resistance defined by an equivalent single-degree-of-freedom oscillator In other words, if the columns do not yield, plastic deformation demand in the beams are expected to be uniform and independent of the beam's strength distribution. A high-rise frame structure without multi-story structural walls, however, does not develop uniform story drift during earthquake excitation.

Figure C4.4 shows some examples of the static and dynamic deformation response (drift angle) of a twelve-story frame building, which was designed using three types of lateral load distribution; i.e., (a) the inverted triangular distribution, (b) the inverted triangular distribution with the concentrated roof-level load (P_t = 0.05 T Q_1), and (c) the inverted triangular distribution with the concentrated roof-level load (P_t = 0.10 T Q_1) A tall frame building designed using inverted triangular load distribution (seismic coefficient shown in Fig C4.4 (a)) developed large beam deformation in the upper stories. The large response at upper stories would be caused by top heavy distribution of earthquake forces (in other words, dynamic effect) during an earthquake motion, which is associated with elastic deformation of columns before formation of the total yield mechanism. Those large responses could be reduced by increasing the beam resistance at upper stories (as shown in Figs C4 4 (b) and (c)) by concentrating a part of earthquake loads at the roof-level However, there has not been a theoretical method established of determining the distribution of beam strength to ensure a uniform distribution of beam deformation and ductility in a structure

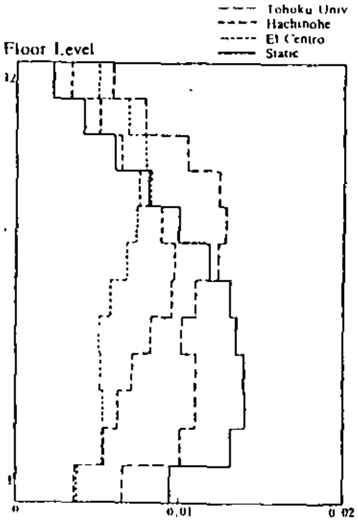
According to a nonlinear earthquake response analysis of a structure, on an average the distribution of lateral forces is similar to that of an inverted triangular distribution, which is similar to the fundamental mode shape, and the maximum story shear force can be calculated by superposing this fundamental mode response with higher mode responses, which are deviated from the fundamental mode response) The lateral forces of deviating components may increase lateral loads in the upper stories at an instance, and then may increase the lateral load at the lower stories at other instances The increase of the upper story lateral load is of significance for the maximum deformation of the structure. The deformation of beams may also be increased by the elastic deformation of columns

Within this guidelines an inverted triangular lateral load distribution is specified as a fundamental design lateral load distribution For a relatively high building, an additional load concentrated at the roof level is specified in order to increase the beam strengths in the upper stories The amount of this additional top story load is determined from the earthquake response analysis in a conservative manner, which can be reduced with a special investigation. Also, other types of load distribution may be employed if uniform story drifts may develop. All other effects that are caused by the higher modes on lateral load distribution are considered in the yield mechanism assuring design when designing the columns and structural walls



(a) Triangular distribution.

(b) Triangular distribution with the concentrated load (P_t = 0.05TQ_1).



(c) Triangular distribution with the concentrated load (P_t = 0.10TQ_1).

Fig. C4.4 Design load distribution and response deformation in a frame structure.

4.3.2 Linear Analysis

The strength distribution at the planned yield hinges should be determined by the analysis that represents a realistic stress distribution at the formation of the total yield mechanism. The reasons are:

(1) to avoid yield hinges formed during a moderate earthquake motion,

(2) to develop a simultaneous yielding at the planned yield hinges to prevent concentrated deformation during a strong earthquake motion, and

(3) to ensure the formation of the total yield mechanism prior to the design limit deformation.

Therefore, a linear analysis with the stiffness deterioration due to cracking or, preferably a nonlinear analysis is recommended. In a linear analysis, to use the secant stiffness at the yield point evaluated for the actual steel arrangement for members with the planned yield hinges and/or the reduced stiffness due to cracking and others is strongly desirable depending upon the stress levels reached under the earthquake design load. Since, however, this procedure is very complicated and in some cases may not be fully justified, the stiffness reduction factor listed in Table C4.1 may be used for simplicity. The effective width of slabs for the evaluation of beam elastic stiffness specified in the "AIJ Standard for Structural Calculation of Reinforced Concrete Structures and Commentary [Ref. 4.1]" can be employed.

The stiffness of members with the planned yield hinges should be carefully estimated since their stiffness is strongly related to the value of the design limit deformation. The stiffness reduction factor for members with the yield hinges planned at both ends should be less than 0.5. The reduction factor should not be reduced intentionally by convenience of design. Therefore, the lower bound is specified as the secant stiffness at the yield point. The stiffness reduction factor will be evaluated when the amount of reinforcement is determined. The reduction factor, however, may be taken not less than 0.3 at the initial stage of design without confirmation.

If a yield hinge is planned only at an end of column such as a first-story column, the use of reduced member stiffness in an elastic analysis may not give a realistic stress and deformation distribution. For an exact analysis, the use of rotational springs at the planned hinge is required. The base of a first-story column may be designed with sufficient amount of reinforcement more than required against the design force. When the structure thus designed is analyzed in a nonlinear range, the base of the first-story column does not yield up to the design limit deformation (building drift angle of 1/100) in a frame structure. Consequently, in a conservative design, an elastic stiffness may be used for a first-story column in a linear analysis for the yield mechanism design.

The elastic stiffness reduction does not significantly change the member force distribution in a frame structure, but both elastic member stiffness and foundation stiffness affect the stress distribution between frames and structural walls in a wall-frame structure. In wall-frame structures, it should be noted that the simplified shear force distribution method such as the Muto's D-value method does not give an accurate shear force distribution. For those structures, a linear analysis using a computer is recommended.

For a member without the planned yield hinges, the uncracked elastic stiffness may be used for simplicity, but use of the reduced stiffness depending on stress levels for both shear and axial loads under design earthquake loads is desirable.

In a frame analysis, all frames can be inter-connected assuming rigid floor slabs, and be analyzed as a structure. In the case when a large shear force is transferred from frames to a structural wall, the in-plane stiffness and resistance of the floor slab should be considered. The effect of transverse members, which are expected due to differential vertical deformations between the wall and frames, may be considered in a linear analysis for the yield mechanism design using a three-dimensional or pseudo coupled model. In this case, the transverse beam should be designed to resist the forces calculated in the analysis.

The linear analysis is desired to be as realistic as possible. The redistribution of calculated forces, however, is allowed to make the steel reinforcement arrangement uniform and construction with ease.

TABLE C4.1 STIFFNESS REDUCTION FACTOR IN A LINEAR ANALYSIS

	Flexural Stiffness	Axial Stiffness	Shear Stiffness
(a) Beam			
Without Hinges	1.0	(1.0)	(1.0)
Yield Hinges	0.3 – 0.5	(1.0)	0.3 – (1.0)
(b) Column			
Without Hinges	1.0	1.0	(1.0)
One Yield Hinge	0.7	1.0	(1.0)
Two Yield Hinges	0.3 – 0.5	1.0	(1.0)
(c) Structural Wall			
Without Hinges	1.0	1.0	0.5 – 1.0
Yield Hinge	0.3 – 0.5	1.0	0.3 – 0.5
(d) Beam-column Connection	—	—	1.0
(e) Slab	0.0	(1.0)	(1.0)

[Note] The stiffness may be taken infinity for (1.0).

4.3.3 Moment Redistribution

If the amount of reinforcement at the planned yield hinges is greater than that required by the linear analysis against the design earthquake loads in the yield mechanism design, the resultant horizontal load carrying capacity of the structure would become larger than that required by the earthquake design loads, resulting from the rounding of the number of reinforcing bars, the limitation of bar size in available, the convenience of using the same bar sizes, the reinforcing bars continuous through a beam-column joint by the reason of easy construction and others. The required amount of top and bottom steel is significantly different with each other at the beam end, since the amount of bottom reinforcement is often governed by the minimum requirement of a tensile reinforcement ratio or that of a compressive to tensile reinforcement ratio. And the ultimate flexural strength of a wall base often exceeds the required design moment. If the strength at the planned yield hinges becomes excessively large as mentioned above, the regions other than the planned yield hinges must be designed for proportionally enlarged design forces in the yield mechanism assuring design that follows the yield mechanism design. If the excess of the strength were large, the yield mechanism assuring design will face some difficulties in arrangement of steel bars, and as a result the design of a structure becomes uneconomical

In order to minimize the overstrength at the planned yield hinges, the redistribution of design moments is allowed; i e , the large design moments at some planned yield hinges can be distributed to other planned yield hinges of the small design moments resulting in an uniform strength distribution There is no definite procedure specified for this redistribution.

The redistributed moments should be examined to satisfy the equilibrium conditions with the design earthquake loads. In the case that the moment is redistributed among members in the story, the amount of redistributed moment may be checked at the story node moments. If the moment is redistributed cross the story levels in the structure, the total work done by the yield moments through a set of virtual displacements should be checked greater than that done by the design earthquake loads through the prescribed virtual displacements

Since a linear analysis may determine a real stress distribution at the formation of the total yield mechanism, it is desirable that the amount of moment redistribution will be as small as possible. The limitation on the amount of redistributed moment is placed. The amount should be allowed within the range to satisfy the conditions (1) through (3) in 4.3 2 "Linear Analysis " A recent study on the moment redistribution of a frame structure [Ref 4.7] has shown that over-reinforcement can be avoided within this limit temporarily placed and the earthquake responses may fall within reasonable values

4.3 4 Deformation Limit

This design guidelines assumes the design limit deformation (the expected maximum inelastic response deformation during a strong earthquake motion) to be 1/100 of story drift angle. Setting the building drift to be the prescribed value at the design earthquake load in the yield mechanism design, the structure is ensured to possess adequate stiffness and to develop the necessary resistance by the design limit deformation indirectly.

Since first the limiting value is intended to ensure the stiffness, and secondly the concentration of deformation in specific limited stories are avoided in the design, the building drift can be defined as the deformation of the centroid of external forces relative to the foundation. The liming deformation is intended for the stiffness control of a super structure The deformation caused by flexibility in the foundation may be ignored, since the foundation stiffness is hard to be evaluated with sufficient accuracy.

It should be noted that the building drift shall be calculated to estimate the yield deformation using the reduced stiffness of the members Therefore, if a linear analysis is used, the stiffness reduction due to cracking should be properly represented in the member stiffness In an actual design process, since the cracked stiffness (exactly speaking, the secant stiffness obtained from the yield point) cannot be defined prior member proportioning, the following simplified stiffness reduction ratios can be used:

(1) elastic stiffness for members without planned yield hinges;
(2) stiffness reduction of 0 5 for members with planned yield hinges at the both ends,
(3) stiffness reduction of 0.7 for members with planned yield hinges at the one end, and
(4) deformation limit decreased to the building drift angle of 1/300 radian for the structure

If the story drift obtained by the above simplified concepts does not meet the deformation limitation, the deformation must be reexamined in detail in the yield mechanism assuring design by either a linear analysis using more realistic member stiffness or a nonlinear static incremental load analysis. Some difficulties in design may be faced when a nonlinear analysis is employed to examine the deformation limitation, since the upper bound strengths are taken for strengths at the yield hinges in the analysis within the yield mechanism assuring design. For the difficulty in the design mentioned above, if the resistance at the deformation limit is evaluated to be greater than the design earthquake loads, the structure is regarded to satisfy the deformation limitation.

4 4 Yield Mechanism Assuring Design (Assurance Design)

4.4.1 Design Actions

Design actions in the yield mechanism assuring design shall be based on a static nonlinear analysis assuming upper bound strengths to develop at the planned yield hinges, in which design forces are magnified by the following factors·

(1) the dynamic effects; and
(2) the concurrency of bidirectional earthquake actions

4.4 2 Non-Linear Analysis

Stresses at formation of the planned yield mechanism shall be calculated by a static nonlinear analysis based on inelastic stiffness properties of members and the following assumptions

(1) Either incremental nonlinear analysis or virtual work plastic analysis shall be used under earthquake loads whose the seismic coefficients are distributed in a triangular shape.
(2) Upper bound strength evaluated for the designed section shall be assigned at the planned yield hinges.
(3) Stress transfer between adjacent frames through diaphragms shall be considered.
(4) If an incremental nonlinear analysis is used, the member stiffness shall be properly evaluated on the basis of elastic stiffness and section as designed. If a yield hinge is not formed until the assuring deformation of the structure, the stresses at the assuring deformation may be used as stresses at the formation of the intended yield mechanism.
(5) If a virtual work plastic analysis is used, the intended yield mechanism shall be assumed. Stresses at column ends other than at column hinges shall be estimated by distributing the sum of beam hinge moments at each joint on the basis of column moment ratios from a linear analysis. Actions in structural walls may be estimated as the difference between the story shear and the sum of column shears.

4 4.3 Magnification due to Dynamic Effect

The dynamic magnification factor on the design moment and shear for columns and structural walls can be given by Eqs.(4.5) and (4.6) unless further special studies have been performed.

$$\omega_{ci} = 1.0 + (\Delta\omega_i / \phi_o) (\beta_{chi} / \beta_{ci}) \tag{4.5}$$

$$\omega_{wi} = 1.0 + (\Delta\omega_i / \phi_o) (\beta_{whi} / \beta_{wi}) \tag{4.6}$$

The higher mode coefficient $\Delta\omega_i$ in Eqs (4.5) and (5.6) shall be given by Eq (4.7).

$$\Delta\omega_i = \begin{cases} 0.25 & \text{for } i = 1 \\ 0.20 & \text{for } 2 \leq i < n/2 \\ 0.20 + 0.10\,(i - n/2) & \text{for } n/2 \leq i \end{cases} \tag{4.7}$$

where the notations denote respectively as follows:

ω_{ci}, ω_{wi} : dynamic magnification factors of columns and structural walls at the i-th story;

ϕ_o : structural strength magnification factor in the yield mechanism assuring design ($= C_{1o} / 0.25$);

C_{1o} : base shear coefficient in the yield mechanism assuring design;

β_{ci}, β_{wi} : ratios of the shear carried by columns and structural walls in the i-th story under the fundamental mode distribution of earthquake forces; and

β_{chi}, β_{whi}: ratios of the shear carried by columns and structural walls in the i-th story under the higher mode distribution of earthquake forces.

4.4.4 Magnification due to Concurrency of Bidirectional Earthquake Actions

Design shears and moments of columns shall be magnified by a factor equal to the sum of the dynamic magnification factor and concurrency safety factor ψ_2. The concurrency safety factor along each principal direction may be taken as 0.10 Design axial loads of columns and structural walls shall be calculated by adding 50 percent of the axial load generated by the orthogonal earthquake forces

4.4.5 Assuring Deformation

Intended yield hinges shall be provided with a deformation capability greater than the assuring deformation Assuring deformation of a member shall be determined by a static nonlinear analysis at the assuring deformation of the structure

[Commentary]

4.4.1 Design Actions

In the yield hinge assuring design, member regions other than at the planned yield hinges are ensured to resist forces higher than the upper bound forces that might possibly be developed in the regions during a strong earthquake motion, and the planned total yield mechanism is assured to be formed In the design guidelines, the fundamental forces at the formation of the planned total yield mechanism are calculated through a nonlinear static analysis under the assumed force distribution along the structure height. The calculated forces can be greater than the design forces in a strong earthquake motion by the reasons in the following

(1) The strength at the yield hinges may be increased by the effects of steel yield strength higher than the nominal strength, strain hardening and increased effective width of floor slabs (over strength).

(2) Lateral load distribution can be different from that assumed in the static nonlinear analysis, which may increase the stress in the columns and structural walls (dynamic effect)

(3) The columns and structural walls are subjected to earthquake motions simultaneously along the two orthogonal directions (concurrency effect of the bidirectional earthquake responses).

The basic design forces for the yield mechanism assuring design (Assurance Design) should be evaluated by a nonlinear static analysis under a monotonically increasing load with the design earthquake load distribution assuming the upper bound strength at the planned yield hinges followed by magnification due to both the dynamic effect and concurrency effect of the bidirectional earthquake motion excitation.

4.4.2 Nonlinear Analysis

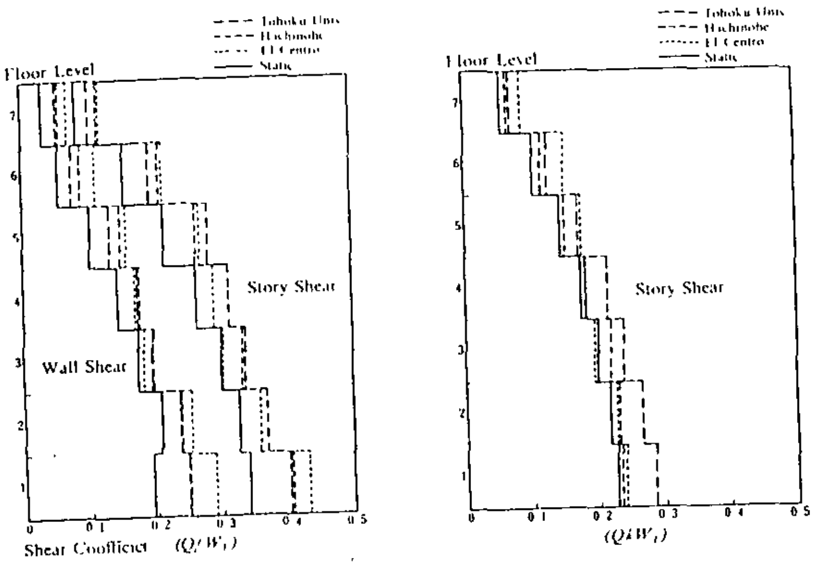
The vibration of a structure forming the total yield mechanism is generally dominated by the fundamental mode (i.e., by the first mode) response during an earthquake. Thus, the basic design forces are calculated by a nonlinear static analysis under a monotonically increasing lateral load with an inverted triangular distribution. The dynamic magnification is defined for the stress distribution obtained from the assumed inverted triangular load distribution.

It is desirable to carry out a static nonlinear analysis using a computer by an incremental analysis with monotonically increasing loads If the total yield mechanism is not formed by the assurance deformation, the horizontal load carrying capacity can be defined as the resistance at the assurance deformation If a simple hand calculation procedure is adopted for a simple frame structure, the story shear may be distributed among the columns proportional to an elastic distribution In a wall-frame structure, the shear distribution between the frames and structural walls may be determined based on the distribution at the yield deformation.

In any case, it is difficult to determine the stiffness and stress at the base of the first-story columns and structural walls with varying axial loads. Design axial loads used in the design of columns should take the bidirectional effect of earthquake responses into account. Since the effect of fluctuating axial loads on the strength of the total structure will be canceled in the tensile and compressive sides, the nonlinear analysis may be conducted along only one direction. Columns that are supposed to be strongly affected by fluctuation of axial loads may be examined by an approximate procedure A hand calculation method, however, for estimating the effect of axial load fluctuation is not available An analysis using a computer cannot give reliable results Engineering judgment will be required in either cases.

4.4.3 Magnification due to Dynamic Effect

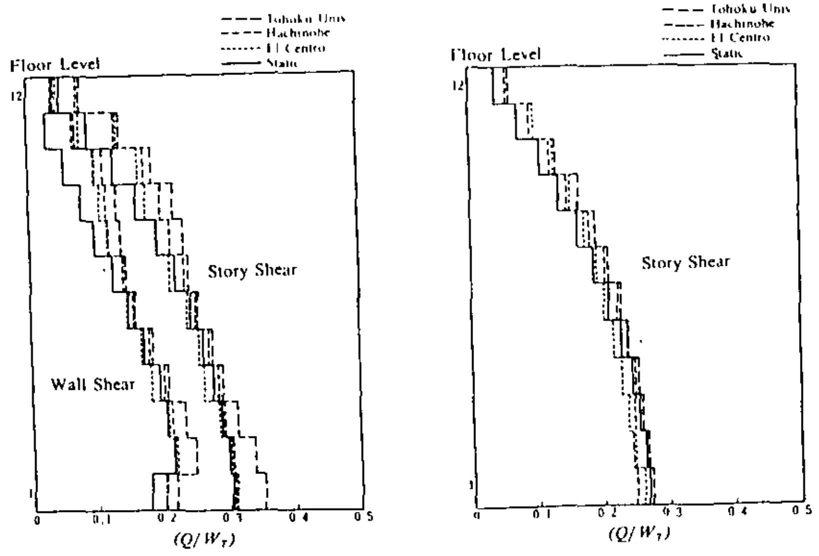
The distribution of response horizontal forces varies with time. The member forces also vary with time fluctuating from those obtained by the static analysis. Figure C4.5 shows the maximum response story shear obtained from a nonlinear earthquake response analysis in comparison with the story shear obtained from a static nonlinear analysis with the inverted triangular lateral load distribution (at the building drift angle of 1/100) The response story shears are generally larger than those obtained from the static analysis. The dynamic amplification factor (one of safety factors), which



Structural wall-frame structure

Frame structure

(a) A seven-story full-scale building



Structural wall-frame structure

Frame Structure

(b) A twelve-story building

Fig. C 4.5 Maximum response story shear.

takes the dynamic effect into consideration, is determined as the ratio of the maximum response story shear to the static story shear

It is difficult to estimate the effect of dynamic amplification for a nonlinear structure theoretically.

In order to make the dynamic effect for a nonlinear structure clear, the response story shears obtained in a nonlinear earthquake response analysis are decomposed into similar vibration modes to elastic vibration modes [Refs 4 8 and 4 9] The vibration mode in an inelastic stage is not identical to that in an elastic analysis Both the difference between the response story shears and horizontal forces obtained from a nonlinear analysis, and the basic horizontal load distribution can be determined from the components of higher vibration modes with use of conventional processes

It is interesting to realize the tendency that the higher mode components that are calculated from the results of a nonlinear dynamic analysis show linear correlation with intensity of the input motion. Both the distribution and amplitudes of higher mode forces along the height cannot be well determined by a theory, while the following two features based on an elastic theory will be of much use in determining the distribution shape

(1) The second mode is considered significant in higher mode forces, in which case the constant mode shape and amplification factor can be assumed.

(2) Base on the sum of higher mode components, the second mode shape and amplification factor can be defined

The number one feature is of much use in the response of a frame structure whose second mode contribution is relatively large, while the number two feature is employed better for the response of a wall-frame structure.

Based on these features to represent higher mode contribution using exclusively the second mode response, the force components of higher modes can be related to both the mass of a structure and intensity of the earthquake motion on the basis of assumption of a constant amplification factor for the second mode. Thus, the higher mode forces can be approximately expressed by the intensity of earthquake motion.

Based on the concept described above, the maximum response story shear of higher mode components can be estimated from the maximum ground acceleration Since the static story shear under the horizontal force distribution of the fundamental mode is limited by the story shear Q_{smax} at the formation of the total yield mechanism, the sum of the maximum static story shear Q_{smax} and the higher mode story shear Q_{fmax} will give the upper bound of the maximum dynamic story shear Q_{dmax}; i.e.,

$$Q_{dmax} \leq Q_{smax} + Q_{fmax} \qquad \text{(Eq. C4.4)}$$

If this upper bound story shear is taken as the design story shear, the dynamic amplification factor from the maximum static story shear Q_{smax} is defined by the following equation

$$\begin{aligned} \omega &= (Q_{smax} + Q_{fmax}) / Q_{smax} \\ &= 1.0 + Q_{fmax} / Q_{smax} \qquad \text{(Eq C4 5)} \end{aligned}$$

The higher mode story shear will be given in proportion to the sum of the dead and live loads (weights) of the structure above the i-th story ΣW_i and the design acceleration level By using the shear coefficient under higher modes ΔC_{hi}, which reveals an envelope of shear distribution along the height under the second and other higher modes of dynamic forces associated with the design acceleration level, the relation is expressed as

$$Q_{fmaxi} = \Delta C_{hi}\ \Sigma W_i$$

and the static story shear Q_{smaxi}, which includes the effects of upper bound strength, can be expressed as:

$$Q_{smaxi} = C_{bi}\ C_B\ \phi_o\ \Sigma W_i$$

where C_{bi} and ϕ_o denote the shear coefficient under the basic mode (C_{b1}=1.0) and the structural strength magnification factor defined as the ratio of the static base shear coefficient (C_{10}) to the constant shear coefficient of C_B (= 0 25), respectively. The formula for the dynamic magnification factor defined in Eq. (4 5) can be derived using the higher mode coefficient.

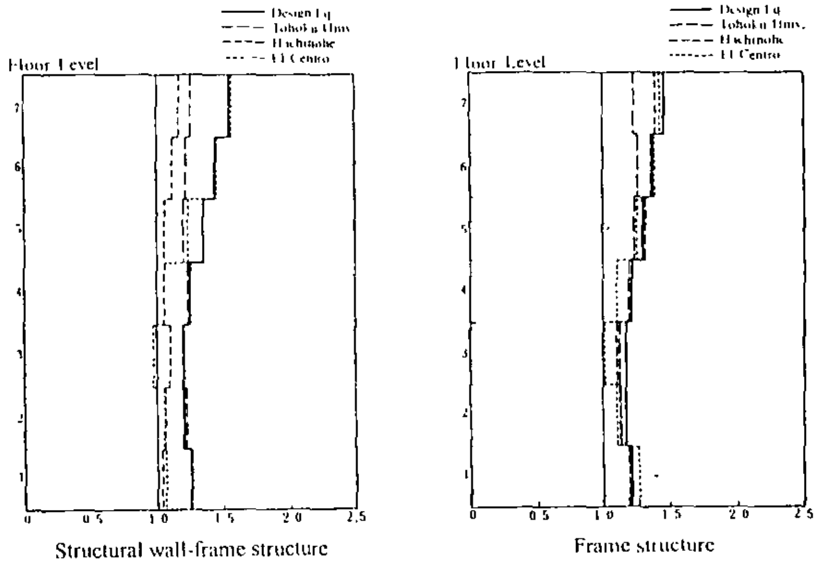
$$\Delta\omega_i = \Delta C_{hi} / (C_{bi}\ C_B)$$

The coefficients related to the dynamic magnification can be derived theoretically by assuming the higher mode shapes and their amplification ratios The expression of Eq. (4 7) in the guidelines is an approximation conservatively derived setting the factors for the second mode amplification ratio of 2.0

This design guidelines proposes the simplified equations (Eqs. 4 5 and 4.6) based on the definition of dynamic amplification factor above mentioned and the results of dynamic analysis In Fig. C4 9, the solid and dashed lines show the dynamic amplification factor defined in the guidelines and the results obtained from the nonlinear response analysis subjected to the recorded earthquake motions, respectively.

The dynamic amplification factor of the shear in each column does not always equal that defined for the total story shear, because the shear carrying distribution ratios among columns vary during an earthquake excitation However, the variation with time among columns can be neglected in design if moment redistribution is taken into account. Therefore, the dynamic amplification factor for each column is taken identical to that derived for the story shear. In general, as for the column moments, the maximum dynamic amplification will be larger than that of the story shear, due to the fluctuation of the inflection point within the column However, even if the dynamic response may cause the column yield moment to be reached, it will occur at one end only within a limited duration An analytical study on dynamic response indicates the evidence that the dynamic amplification factor for moment, which is defined as the ratio of the higher moment at the top or bottom end to the design moment, is almost equal to the dynamic amplification factor obtained from the story shear. The factors for the moment are given similar to those for the story shear within this guideline, through which a partial column sidesway yield mechanism can be prevented

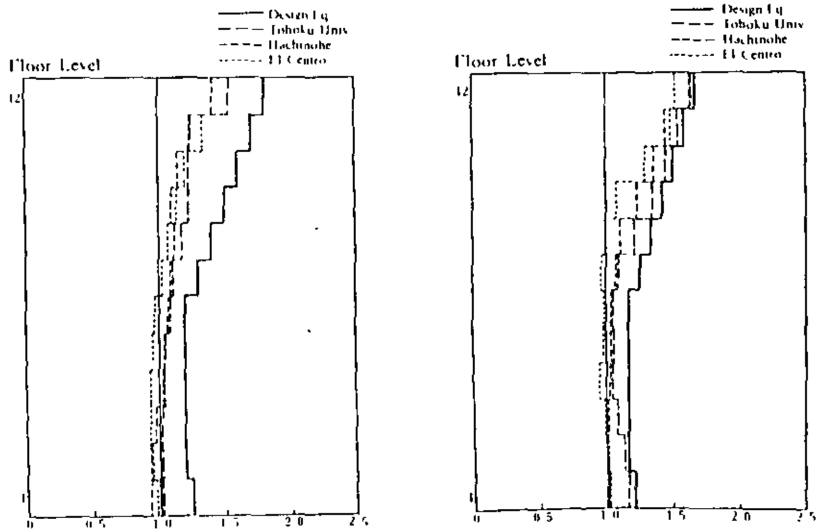
Due to the effect of higher modes of vibration, the axial load in an exterior column induced during earthquake excitation might be less than the sum of beam shear forces calculated from the



Structural wall-frame structure

Frame structure

(a) A seven-story full-scale building



Structural wall-frame structure

Frame Structure

(b) A twelve-story building

Fig. C 4.9 Dynamic response amplification and design dynamic amplificaion factors.

upper bound yield moments. A reduction factor for axial loads, however, will not be introduced, since the effect of axial load fluctuation is of less significance for the structures not higher than 45m.

In a wall-frame structure, the structural wall carries most of the shear caused by the higher mode response, even if the wall component is small to the total structural size. The distribution ratio of the higher mode shear to the column and wall can be estimated by decomposing the dynamic shear of column and wall in a similar manner. Figure C4.10 shows the ratios of the wall and column shear forces caused by the higher mode. The ratio of shear carried by the structural walls under the basic mode of earthquake forces is about 60% at the first story, and decreases in the upper story levels. A large part of the higher mode shear, up to 80%, is carried by the structural walls and the distribution of the ratios along the height of the structure is almost uniform.

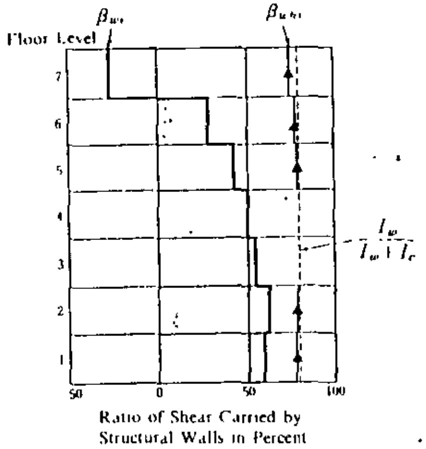
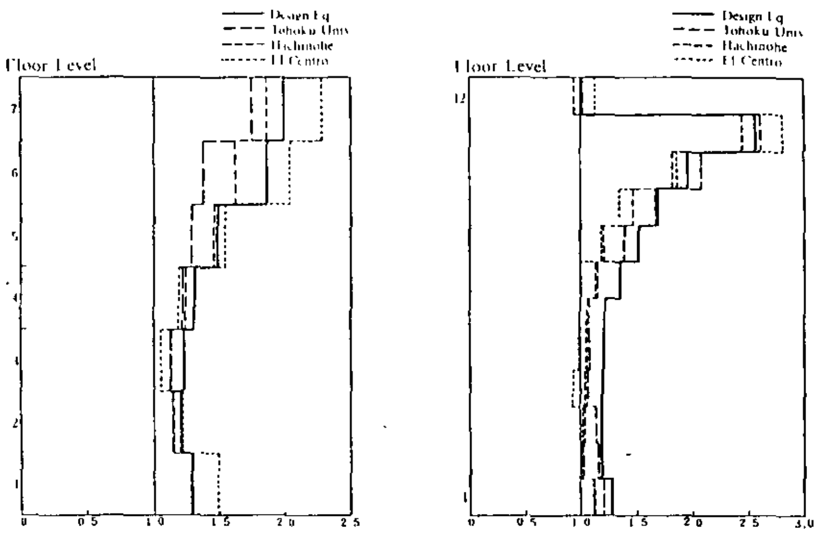


Fig. C 4.10 Shear distribution associated with higher mode components.

Referring to the dynamic analyses of a structure with different amount of structural wall components, the distribution ratios of the higher mode story shear to columns and structural walls at the i-th story level, β_{chi} and β_{whi}, can be given by the following equations:

$$\beta_{chi} = I_c / (I_c + I_w) \qquad \text{(Eq. C4.6)}$$
$$\beta_{whi} = I_w / (I_c + I_w) \qquad \text{(Eq. C4.7)}$$

where I_c and I_w signify the sum of the stiffness of the columns and structural walls in the story under antisymmetric bending, considering shear deformation, respectively. Figure C4.10 shows the ratios given from the equations. The design dynamic amplification factor for structural walls and that obtained from dynamic response analysis are indicated in Fig. C4.11.



(a) A seven-story building

(b) A twelve-story building

Fig. C 4.11 Dynamic magnification on responses of story shears in structural walls.

If the shear failure of the wall is prevented, yielding of columns might be allowed, because the overall yield mechanism of the structure is ensured by the structural wall. It is desirable, however, to ensure the columns not to form yield hinges, since it is with more ease to design the beams as ductile members than the columns that carry axial compressive loads.

The dynamic amplification factor for the moment of a structural wall does not necessarily correspond to that for the shear above obtained, and it should be formulated rationally considering the moment distribution of the beam and structural wall components. The amplification factor for the moment, however, can be taken identical with that for the shear for simplicity, because an occasional flexural yielding of the structural wall will not be critical in seismic performance within an overall yield mechanism.

4.4.4 Magnification due to Concurrency of Bidirectional Earthquake Actions

The yield surface of a column under bidirectional bending generally describes a circle, an ellipse or some convex shaped curve. The maximum column moment or shear in the beam-yielding mechanism could reach the resultant of the maximum within the plane. The ratio of the maximum amplitude along the diagonal direction to that along the principal directions could reach approximately the square root of 2 in the beam-yielding mechanism for the case when the concurrency of bidirectional earthquake forces is considered. The earthquake induced axial force in a column positioned at the corner of a structure by the two-way action could reach twice as large as that by the one-way action

The design forces given as the combination of the overstrength forces from each direction will be large enough to realize a beam yield mechanism under bidirectional earthquake forces A non-linear dynamic analysis against bidirectional ground motions indicates that there is a low probability of concurrence of the overstrength forces along the two directions simultaneously The combination of the overstrength forces from two directions, with other magnification factors such as those for overstrength and dynamic effect, will be regarded conservative in design. If probability distribution for those factors were determined, the probability of structure failure could be evaluated. At present, however, it is very difficult to fix those probability distributions for various types of structures, earthquakes, seismic intensity levels and other factors for responses of a structure

This design guidelines determines the forces acted from structural frames placed in the transverse direction invariant. These concurrent forces are added to the static forces, which are regarded independent of the dynamic amplification effect. Further discussions are needed on this decision and the fixed figures.

Based on the results of the research on a full-scale 7 store building for the maximum overturning moment when excited in both directions. The forces acted from the transverse direction are about 50% of the maximum forces from the direction under consideration in the case of the real EL Centro motion, which reveals small concurrent effects, but in the cases of other real earthquake motions the ratio of forces from the transverse direction to the longitudinal direction comes to the range from 70 % to 100 %. Within the design guidelines, the use of 50 % of the forces from the transverse yield hinge mechanism obtained from a static non-linear analysis is proposed as the concurrency of bidirectional excitation.

4.4.5 Assuring Deformation

In the yield hinge mechanism design, both the intended yield mechanism of a structure and the deformation capacity of constituent members with yield hinges should be assured.

This design procedure should be included in ductility design. A part of this procedure will be conducted in the yield mechanism assuring design, since a definite methodology of evaluating the required ductility and the ductility capacity of members has not yet been established. The ductility capacity of members will be assured by taking its calculated shear strength smaller than the smallest shear capacity empirically obtained (refer to the Chapter 6 : Design for Shear and Bond).

For a given earthquake motion, the required deformation capacity of the members could be determined But at the present time, both the characteristics and intensity of future earthquake motions cannot be predicted sufficient for design, and it is considered adequate to set the assurance deformation larger than that obtained from dynamic response analyses The design assurance deformation can be calculated based on a static non-linear analysis. Deformation angle is given for every story level by setting larger deformation than the design limit deformation, not setting the relative deformation of the centroid of external forces from the foundation (equivalent deformation of equivalent one degree of freedom system).

The building deformation can be estimated by the EODF system For the deformation of each constituent member, however, the dynamic effect that causes deformation concentrated in limited specific members should be considered. This effect is significant for design in general, and in some cases the design procedures themselves may be modified The member deformation obtained from a static analysis will be of less significance for design. An another procedure can be revealed significant to estimate the assurance deformation for constituent members based on the geometrical relation assuming the column and structural wall to be rigid and using the story drift limits of assurance deformation given in the design guidelines.

(1) For beams : 1/50 of deflection angle
(2) For columns : 1/67 of deflection angle
(3) For structural walls : 1/75 of deflection angle
(4) For boundary beams : 1/40 of deflection angle

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CHAPTER 5 : DESIGN FOR FLEXURE WITH/WITHOUT AXIAL LOAD

5.1 Scope

Provisions of Chapter 5 shall apply for design of members subjected to the flexural loads or those combined with axial loads.

[Commentary]

The provisions of this Chapter shall apply to the design of members subjected to the flexural loads or to the combined flexural and axial loads such as beams, columns, shear walls and piles that resist against earthquake forces. A floor slabs and a sub-beam whose design are dominated by permanent loads are out of the scope of this design guideline. The "Standard for Structural Calculation of Reinforced Concrete Structure" (denoted as the current standard in the following within this chapter) provides the design procedure for these members

5.2 Design Method

5.2.1 Principles

(1) Design of the member subjected to the flexural loads or those combined with axial loads shall be based on the basic assumptions given in Section 5.2 2; the equilibrium condition of forces and the compatibility condition of strains

(2) Design shall consider the largest combination of flexural and axial loads

[Commentary]

(1) In the second stage of the current design, in which the lateral load carrying capacity and the design shear forces are calculated, the simplified equations for ultimate flexural strength of column and beam are generally used. In this design guideline, however, the design of a member subjected to the flexural or the combined flexural and axial loads is based on an inelastic flexural theory satisfying the equilibrium of forces and compatibility of strains, in order to assure the yield mechanism and yield assurance designs. Recently, computers are used so commonly that the design of member, even of a T-shaped section or with multi-layered steel bars, can be done easily based on an inelastic flexural theory

(2) A column is generally subjected to the combined flexural and axial loads, and the ratio of these combination always changes during an earthquake. The important factors for the design of a column are the fluctuation of axial loads and that of the design moment of non-hinged member during an earthquake. In the yield hinge mechanism design, the axial load of a column should be combined with the gravity load and the overturning moment due to earthquake excitation. In the yield assurance design, the fluctuation of axial loads and the fluctuation of the design moment of non-hinged column should be considered The corner columns should be designed against the four types of combination of design loads, considering the tensile and compressive axial loads due to the overturning moment produced within longitudinal and transverse frames

5.2.2 Basic Assumptions in Calculation of Ultimate Flexural Strength

(1) Strains in reinforcement and concrete shall be assumed directly proportional to the distance from the neutral axis.

(2) Stress-strain relationship of reinforcement shall be linearly elastic in both tension and compression to the material strength given in Table 2 1. For strains greater than that corresponding to the material strength, stress in reinforcement shall be equal to the material strength.

(3) Nonlinearity of the stress-strain relationship of concrete shall be considered.

[Commentary]

(1) In a calculation of the ultimate flexural strength, a plain remaining plain is assumed. The calculated strength based on this assumption matches well to the experimental result, even if this assumption would not be good for the cracked section. However, the member with thin width and deep depth, and with the shear deformation not negligible, would have a larger flexural strength than that based on plain remaining plain, especially in the case of compression toe failure. The evaluation of the strength for the member expected to be compression failure in its toe under the combined flexural and shear forces has not been established yet, that is the research item from now. Then this guideline restricts the shape of a member to satisfy the assumption of plain remaining plain.

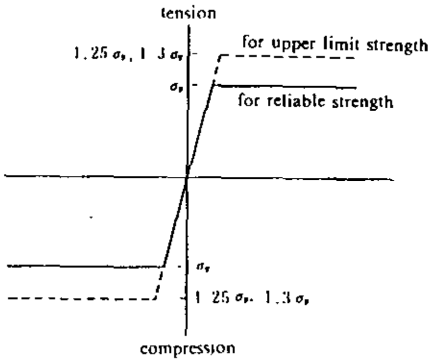
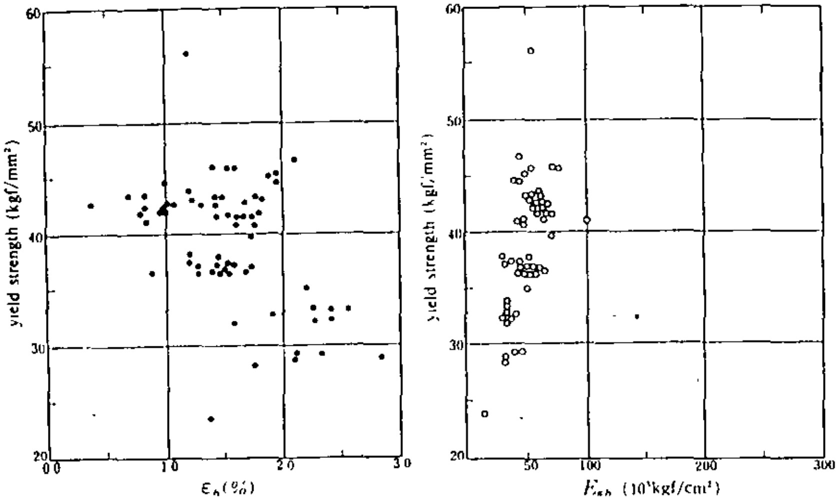


Fig. C 5.1 Stress-strain relationship of a steel bar.

(2) The stress-strain relationship of a steel is assumed to be the elasto-plastic as shown in Fig. C5.1 The modulus of elasticity of reinforcement is assumed to be 2 1×10^6 kgf/cm^2. That obtained from material tests for the deformed bars reveals a little less than 2.1×10^6 kgf/cm^2, but the effect of Young's modulus is of little significance on the ultimate flexural strength. Figure C5 1 shows no strain hardening effect because there is a little possibility for reinforcement to go into that region

within the assurance deformation of a structure. As for the member that is forced a large deformation such as a boundary beam with short clear span, however, it is desirable to estimate appropriately the effect of strain hardening on the ultimate flexural strength.

The strain at strain hardening (ε_{sh}) and stiffness after that (E_{sh}) scatter very widely as shown in Fig. C5 2, and it is difficult to fix these values, but it is required to consider the strain hardening effect when the strain of reinforcement is expected to be over 1%.



(a) Strain at strain hardening (b) Stiffness at strain hardening.

Fig. C5.2 Strain ε_{sh} at the strain hardening and hardening stiffness E_{sh}.

(3) Many stress-strain models for compressive characteristics of concrete are proposed. The diagrams in Fig. C5 3 are modeled from an uniaxial compression test, which does not, however, correspond to real stress combination in members. At the analysis of the member subjected to flexural moment, the stress v.s. strain model of concrete presented by a parabolic and a linear curves as shown in Fig C5.3(a) or by the "e-function" curve as shown in Fig. C5.3(b) are generally used, and the analytical and the experimental results match well to each other. ACI adopts the equivalent compressive stress method as shown in Fig. C5 4 in which an average stress of 0.85 σ_B and an effective depth of $\beta_1 x_n$ are assumed for non-linear stress distribution with a rectangular shape.

A stress-strain relationship of concrete does not matter when the ultimate flexural strength of a member is dominated by the yielding of tensile reinforcement, but it is necessary to use its accurate relationship when a stress at the compression toe dominates the ultimate flexural strength of the member.

The effect of a confinement on the compressive characteristic of concrete has not been generally formulated yet, but there are many references on that

Tensile strength of concrete is assumed to be zero.

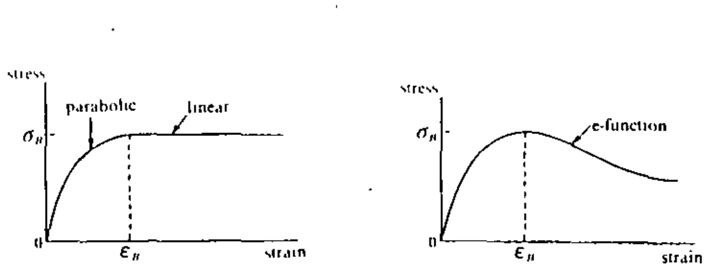


Fig. C 5.3 Examples of the stress-strain curve for concrete.

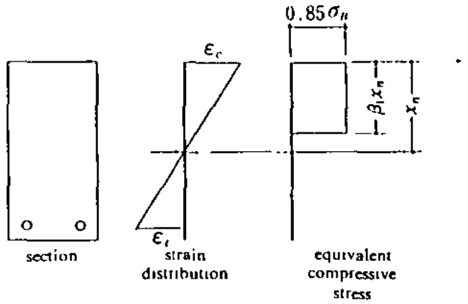


Fig. C 5.4 Equivalent rectangular stress block in the ACI code.

5.3 Reliable Flexural Strength

Reliable flexural strength shall be based on the following assumptions.

(1) Strain at extreme concrete compressive fiber shall be assumed equal to 0.003. Material strength of reinforcement given in Table 2.1 shall be applied for computation of reliable flexural strength.

(2) Tensile reinforcement within effective width of slab may be included in tensile reinforcement at beam top.

(3) Stresses intermediate reinforcement may be considered, if necessary.

[Commentary]

(1) Relationship between moment and curvature of a reinforced concrete member which yielding of tensile reinforcement occurred primarily is schematically drawn in Fig. C5.8 After the Y-point, yielding of tensile reinforcement followed by cracking, an increment of flexural strength is small. The compressive strain of concrete increases with the increment of the tensile strain of reinforcement after the Y-point, where the deformation also increases The U-point on the diagram of Fig. C5 8 denotes the point of the compressive strain of 0 003, and the flexural strength at this point is defined as the ultimate flexural strength in this guideline A beam or a column not subjected to high axial loads maintains its resisting moment after the U-point until the M-point, that is the maximum strength But this guideline deals with the stable flexural strength up to the U-point.

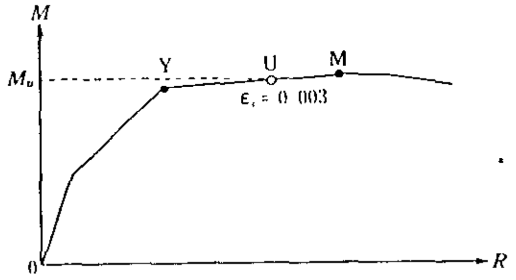
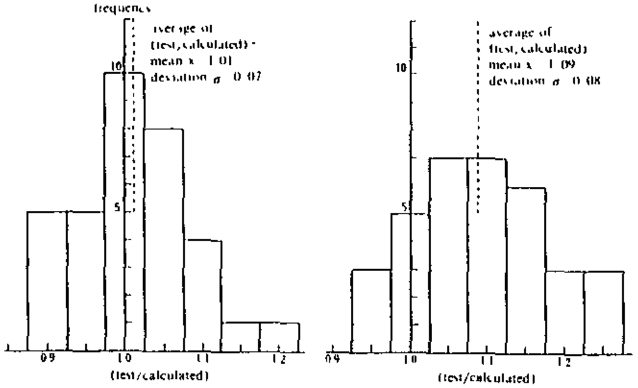


Fig. C5.8 Schematic relation between flexural moment and curvature.

(2) Figure C5.9 shows the histogram of the ratio of the flexural strength by an experiment to that by a calculation about T-shaped beams with yield hinges at both ends. Steels within the effective width of a floor slab are taken into account by the calculation. Figure C5.9(a) shows the case that the average strength of reinforcement obtained from a material test are used as the yield strength, and Fig. C5.9(b) shows the case that the strength of reinforcement specified in JIS (Japan Industrial Standard) are used as the yield strength. These figures indicate that the calculated strength based on the average yield strength of reinforcement gives the average of the ultimate flexural strength, and that on the nominal yield strength of reinforcement by JIS gives the lower bound of the ultimate flexural strength.

The same results were reported about the average ultimate flexural strength of columns conducted under the national research project on the short length columns during the period from 1972 to 1977

This guideline does not introduce the strength reduction factor, because the lowest material strength considering its deviation is given in Chapter 2, and the ultimate flexural strength based on it is expected to give the lowest bound of itself as well as its nominal strength is expected to give the lowest bound of the reliable flexural strength



(a) σ_y : average value of material tests

(b) σ_y . nominal value by JIS (reliable strength)

Fig. C5.9 Histogram of the ratios of empirical to calculated values of flexural strength.

(3) The deformation at the reliable flexural strength should be smaller than the design limit deformation, because it is expected that the moment at the hinge planned must reach the ultimate flexural strength up to the design limit deformation Figure C5.10 illustrates the sketch of the lateral force and displacement relations on three different types of members. The member indicated by the line (a) can be expected the reliable flexural strength up to the design limit deformation R_d, but the member indicated by the line (c) can not be In the case of line (b), the almost same strength as the reliable flexural strength can be expected because the flexural yielding will occur before the design limit deformation R_d

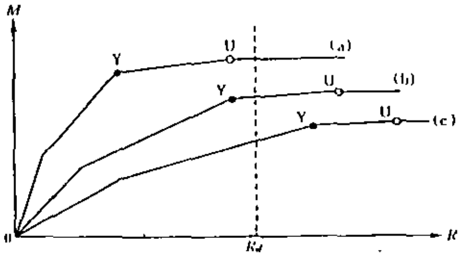


Fig. C5.10 Relationship between ultimate strength and limit design displacement.

Under the assumption of a curvature distribution as shown in Fig C5.11, the deformation of a member R and the shear span ratio a/d have the relationship with each other described in Eq. C5 1 Equation C5.1 is modified to Eq C5.2 by adding the condition of the occurrence of the flexural yielding up to the member's angle of R_d corresponding to the design limit deformation

$$R = (\varepsilon_t + \varepsilon_c)(a / d) / 3 \qquad \text{(Eq C5.1)}$$

$$(\varepsilon_y + \varepsilon_c)(a / d) \leq 3R_d \qquad \text{(Eq. C5 2)}$$

where ε_t : strain of the tensile reinforcement,
ε_y : strain at the yielding of tensile reinforcement, and
ε_c : strain at the extreme compression fiber at the flexural yield strength

Figure C5.12 shows the relationships of $(\varepsilon_y + \varepsilon_c)$ and (a/d) on the condition that R_d is assumed to be 1/150. Assuming that the strain at yielding of the tensile reinforcement is 0.002 and the compressive strain at the extreme fiber is 0 003, ε_t is greater than ε_y at R of 1/150, so that the reliable flexural strength of a member can be obtained before the design limit deformation under the condition that a/d is smaller than 4.

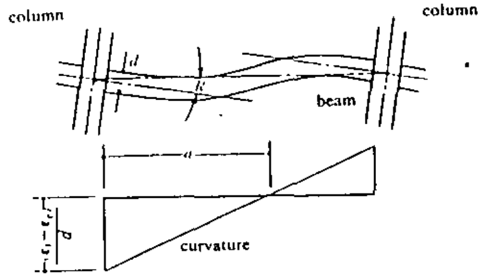


Fig. C5.11 An assumption of curvature distribution along a member.

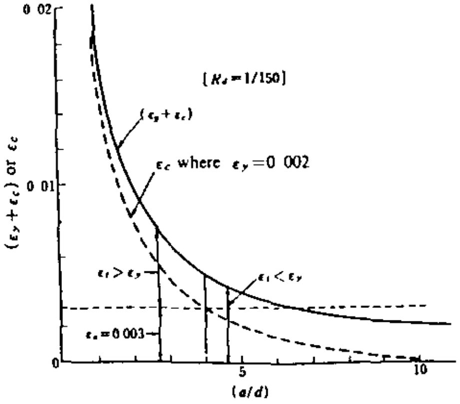


Fig. C5.12 Relation between $(\varepsilon_y + \varepsilon_c)$ and (a / d).

(4) A floor slab cast together with a beam works together as the flange of beam, and the flexural strength of a beam with a floor slab shows higher than that without a floor slab According to the experiment of beams with floor slabs, some longitudinal reinforcement in the floor slab near the beam yield at the almost same time as the tensile reinforcement in the beam, and the yielding of reinforcement in the floor slab extends as increasing of a deformation, and finally all of longitudinal reinforcement in the floor slab yield at the maximum strength

The reliable flexural strength of a beam is the base of the lateral load carrying capacity of a structure, and is the minimum required strength for a beam In order to evaluate the ultimate flexural strength more accurately, an adequate amount of reinforcement in a floor slab should be counted in the calculation for reliable flexural strength of a beam The reinforcement within the effective width of a floor slab specified in the current standard may be counted into the reliable flexural strength if they have enough development length.

(5) The reinforcement placed in the middle part of a shear wall or a wall girder resist against a bending moment in proportion to their strains if they would have enough development length as well as the column In these members, after the yielding of the longitudinal reinforcement concentrated at the extreme tensile end of a member section, some reinforcement placed in the middle part of the member section yield, then the member reaches almost its yield strength. The strength at that time is given considering the contribution of the reinforcement in the middle part of a member section. Shear wall, especially, have a lot of vertical reinforcement in the middle parts It is reasonable to consider these reinforcement in evaluating their reliable flexural strength.

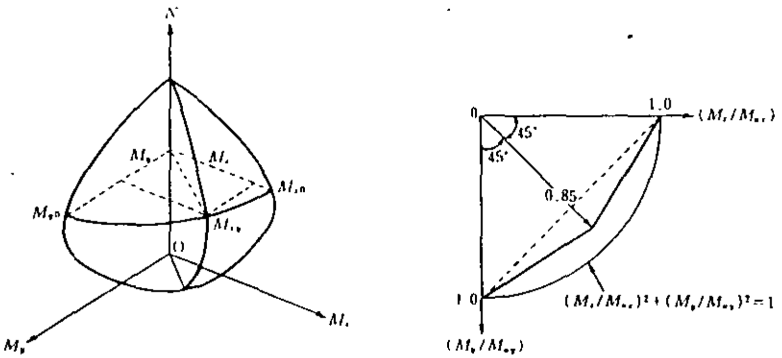
In this guideline, the ultimate strength is given when the compressive strain of concrete is 0.003 at the extreme fiber As for the ultimate flexural strength of a shear wall, however, many test results showed larger strain than 0 003 at the ultimate flexural strength. The ultimate compressive strain of 0.003 may be assumed at the center of a boundary column in compression side for the calculation of the ultimate flexural strength of shear walls

(6) Yield strength of a column subjected to biaxial bending is generally lower than that to uniaxial bending as shown in Fig C5.13. The yield strength diagram would be affected by a cross-sectional shape, an amount of flexural reinforcement, their arrangement, material strength, and axial force. Under the constant axial force, the correlation between the ultimate flexural strength in both X and Y directions are often described as Eq C5.3.

$$(M_x / M_{ux})^{\alpha} + (M_y / M_{uy})^{\alpha} = 1 \qquad \text{(Eq C5 3)}$$

The value of α is generally assumed to be less than 2 and is related to the axial force level. Figure C5.13(b) shows an proposal of the biaxial yield strength diagram whose relationship is assumed to be bilinear and the ultimate strength on 45 degree axis is 0.85 times the strength along the X or Y direction.

The reliable strength design of a column may be performed on each direction independently, because the design moment and the axial force are magnified already in Chapter 4 considering the occurrence of a bidirectional earthquake action. If any flexural design are necessary on any other directions than the principal axis, the flexural strength should be calculated on the direction subjected to the specified forces



(a) Sketch of the yield surface (b) An approximation

Fig. C5.13 Yield surface of a column subjected to bidirectional bending.

5.4 Upper Bound Flexural Strength

Upper bound flexural strength shall be based on the following assumptions:

(1) Strain at extreme concrete compressive fiber shall be assumed equal to 0.003. Material strength of reinforcement given in Table 2.1 shall be applied for computation of upper bound flexural strength

(2) Tensile reinforcement within twice the effective flange width of slabs and walls shall be considered as tensile reinforcement.

(3) Stresses in intermediate reinforcement and all other reinforcement effective for flexural resistance shall be considered

[Commentary]

The upper bound of the ultimate flexural strength which could be expected in the yield hinge region up to the assurance deformation is defined as the upper bound flexural strength The strength deviation of reinforcement in a higher side, the extension of the effective width of a flanged member, the existence of some reinforcement non-calculated, and the effect of a strain hardening of reinforcement in the member with a short span cause the overstrength than the reliable flexural strength It is necessary in the assurance design to estimate the upper bound flexural strength in order to prevent a member from shear failure and to assure the yield hinge mechanism even if the ultimate flexural strength in the yield hinge region would become higher due to the factors mentioned above

The overstrength of reinforcement described in Chapter 2 is used. As for the effective width of a flange, the width twice as large as that assumed to compute the reliable strength should be considered. Although some test data show that all reinforcement in a floor slab yield under a large drift, in this design guideline all reinforcement within full width of a floor slab might not yield up to the assurance deformation The intermediate reinforcement and all other reinforcement effective for flexure should be considered

5.5 Axial Load Limitation in Yield Hinge Region

In the yield hinge region of columns and structural walls defined in Section 9.2.3, design axial loads shall satisfy the following limitations

(1) Axial load in a column shall satisfy Eq.(5.1):

$$-k_2 A_g \sigma_y \le N_c \le k_1 A_c \sigma_B \qquad (5.1)$$

where N_c : axial load in the yield mechanism assuring design, positive in compression, in kgf,

A_c cross sectional area of column, in cm^2;

A_g : gross area of longitudinal reinforcement in column, in cm^2;

σ_B : compressive strength of concrete, in kgf/cm^2; and

σ_y : material strength of reinforcement for reliable strength given in Table 2.1, in kgf/cm^2.

Coefficient k_1 for compressive load shall be 1/3. The value may be increased to 2/3 if confining reinforcement is placed as specified in Chapter 9. Coefficient k_2 for tensile load shall be 3/4.

(2) Axial load in a structural wall shall satisfy Eq.(5.2):

$$N_w \le k_3 A_{core} \sigma_B - A_{ws} \sigma_{wyu} \qquad (5.2)$$

where N_w : total axial load acting on the wall in the yield mechanism assuring design, positive in compression, in kgf;

A_{core} : core area of boundary column on compression side of a wall, in cm^2;

A_{ws} : area of vertical reinforcement in the wall panel, in cm^2; and

σ_{wyu} : material strength of vertical reinforcement in the wall panel for upper bound strength, in kgf/cm^2.

Coefficient k_3 shall be 2/3 The value may be increased to 1.0 if confining reinforcement specified in Chapter 9 is placed in the boundary column.

[Commentary]

(1) An ultimate flexural strength and a deformation capacity after a flexural yielding of a column subjected to axial and flexural forces are significantly affected by the axial force level, and the less deformation capacity could be shown under the higher axial force. In order to maintain a flexural strength and a deformation capacity after flexural yielding, the axial force of a column should be less than $A_c\sigma_B/3$. However, the doubled axial force level $2A_c\sigma_B/3$ might be allowed under the condition that hinge region should be enough confined according to Chapter 9 to prevent compressive reinforcement from buckling or concrete from spalling-off.

A corner column would be subjected to a large axial load as close to $2A_c\sigma_B/3$ because of a bidirectional earthquake action. It would be necessary to have the area of such a corner column large enough in order to reduce its axial load level

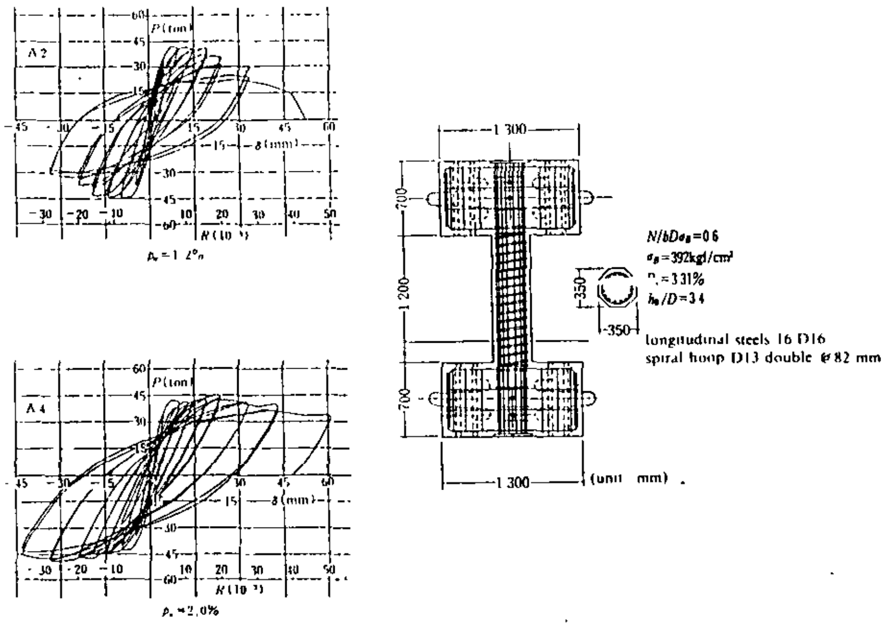
Figure C5.17 shows the result of a column test subjected to very large axial loads Figure C5.17(a) shows the effect of the amount of lateral reinforcement p_w with the constant axial load of $0.6A_c\sigma_B$. Figure C5.17(b) is the test result in which the axial load changes from $0\ 75A_c\sigma_B$ to $-0\ 25A_c\ \sigma_B$ proportional to the drift Both test results show that good ductility could be obtained by enough confinement.

(2) An exterior or a corner column in multi-story frame structure would be subjected to tensile forces during an earthquake. The column subjected to tension and bending shows very ductile behavior because the compression strain of concrete decreases If all vertical reinforcement in a column would yield by tension forces under an earthquake motion, they would be easy to buckle by following compression forces, because the stiffness of all vertical reinforcement decrease due to "Bausinger Effect" There are few tests about such a behavior This design guideline restricts the tensile force to column not larger than 0.75 times the member's tensile yield strength.

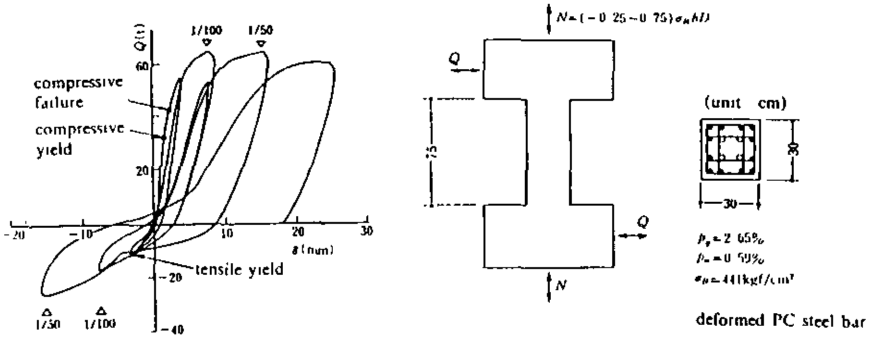
(3) A shear wall has a larger lateral stiffness than a frame structure, and its yield deformation is very small, then a large inelastic deformation after yielding is required Some researches show that the ductility of a flexural type shear wall depends on the strength and ductility of its compression zone including the boundary column Figure C5 18 shows the relationship Between R_u (drift at 80% the maximum strength) and $\bar{\sigma}_c$ (total axial forces divided by axial strength of a boundary column) It indicates that the drift of more than 1/50 could be expected for the shear walls with their $\bar{\sigma}_c$ of less than 2/3.

Figures C5 19 show the test results of shear wall in which main parameter is the area of the boundary columns. The shear span ratios were changed during the test of a specimen The test specimen W1 has a lateral reinforcement p_w of 1.1% in the boundary column and is the prototype The specimens W2, W3 and W4 have p_w of 1 6%, and also the wall panel of W3 is thicker than others. The area of the boundary columns of W4 is bigger than others These test results show that the stable response would be expected up to the drift of 1/50 with shear span ratio of more than 1 5 The axial load levels of the boundary columns were changed during the test, and they were close to the value obtained by Eq. 5 2.

The left side in Eq. 5.2 indicates the total axial loads consisting of a dead load sustained by a shear wall and an axial load due to overturning moment in a plane and a transverse directions at the yield mechanism assuring design. The first term of the right side in Eq. 5.2 is the axial strength of boundary column that is defined as the product of A_{core} (inside of lateral reinforcement as shown in Fig C5.20) multiplied by concrete strength. The second term is a tensile yield strength of the vertical reinforcement in the wall panel. When the amount of the vertical reinforcement in the boundary columns are very different among each other, the second term should take all the vertical reinforcement in wall and boundary columns into account



(a) Effect of p_w (σ_o = 0.6 σ_B)



(b) Fluctuation of axial force.

Fig. C5.17 Experimental results of a column with high axial loads.

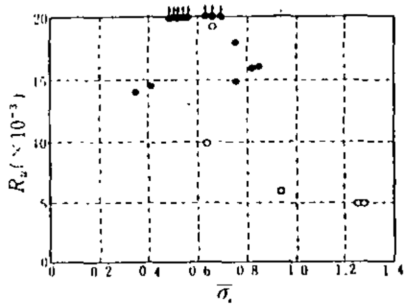
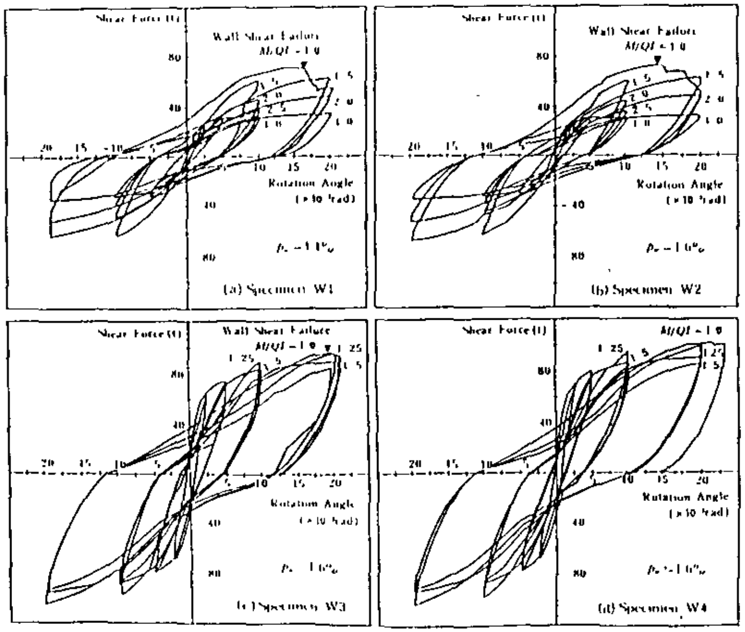


Fig. C5.18 Axial stress of a boundary column $\bar{\sigma}_c$ and the limit deformation R_u.



(a) Specimen W1

(b) Specimen W2

(c) Specimen W3

(d) Specimen W4

Fig. C5.19 Experimental results with various size and confinement of a boundary column.

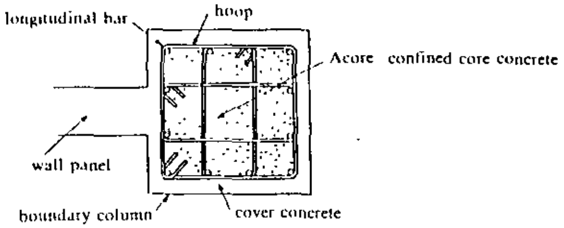


Fig. C5.20 Cross-sectional area of confined core concrete A_{core} in a boundary column.

5.6 Structural Requirements

5.6.1 beams

(1) Sectional Shape

Beam width shall be not less than 25 cm. In a yield hinge region, beam width shall be not less than 1/4 times beam depth.

(2) Longitudinal Reinforcement

1) Longitudinal reinforcement shall be deformed bars of size equal to or larger than D19.

2) Tensile reinforcement ratio in a yield hinge region shall be not greater than 0.025 including effective slab reinforcement. Ratio of total area of compressive reinforcement to that of tensile reinforcement shall be not less than 0.5. The tensile reinforcement ratio pt is defined as ratio of the total area of tensile reinforcement to the product of beam width and effective depth.

3) Tensile reinforcement, as a general rule, shall not be placed in more than two layers.

5.6 2 Columns

(1) Sectional Shape

Minimum dimension of column shall be not less than 40 cm. Ratio of section dimensions of long side to short side shall be not more than 3.0 in a yield hinge region.

(2) Longitudinal Reinforcement

Longitudinal reinforcement shall be deformed bars of size equal to or larger than D19.

5.6.3 Structural Walls

(1) Sectional Shape

1) Cross sectional shape of a structural wall, as a general rule, shall be of I-shape with boundary columns on both sides.

2) Thickness of wall panel shall be not less than 15 cm, and also not less than 1/20 of clear height of the wall panel.

(2) Placement of Reinforcement

1) Reinforcement shall be deformed bars of size equal to or larger than D10, and placed equal amount in the vertical and horizontal directions

2) Reinforcement shall be placed in double layers in a yield hinge region.

(3) Opening within the Wall Panel

If an opening must be placed in a yield hinge region of wall panel, the location shall be at the center in width, and the size shall not affect the monolithic behavior of the wall

[Commentary]

(1) This section mentions the structural requirements about the shape and the dimension of the members, the size and the reinforcement ratio, and etc. The requirements are not necessarily based on the test or analytical data but based on an engineering experience and judgment, and the example of the foreign design codes, especially considering on the keeping ductility and the easy construction.

(2) As for the width of beam, its minimum width is 25 cm that makes easy to cast concrete and to place a shear reinforcement preventing buckling of a compression reinforcement, bond failure of a tensile reinforcement. The ductility of a beam is so emphasized in this design guidelines because it is expected to form a yield mechanism that it would be the best for the width of a beam to be as large as possible.

Some foreign codes have the limitation of a beam width to make sure the anchorage of flexural reinforcement in a beam into a column. But the width of a column is usually larger than the width of a beam in the Japanese building construction and the longitudinal reinforcement in the beam is placed inside of the longitudinal corner reinforcement in the column at beam column joint, then there is no upper limit about the width of beam

The limitation of the beam width not less than 1/4 times the beam depth is also from the view of the ductility Too large ratio of a beam depth to a beam width would make some complicated problems such as a member's buckling or an applicability to a frame model. Shear deformation would affect on an assumption of plain remaining plain when a beam span would be short relatively to a beam depth.

The shear span ratio should be taken into account by the design in Chapter 6, so that there is no limit about the shear span ratio on flexural performance, and thus only the ratio of a beam width to a beam depth is specified in this section

(3) Column size should not be less than 40 cm, and the ratio of the longer size to a shorter one should not be larger than 3. In the case of a wall-column structure which has big ratio of the column depth to column width, it would be necessary to devise an special arrangement of lateral reinforcement in order to confine the compressive zone as shown in Chapter 6, shear design, so there is no limit about the shear span ratio of a column in this section.

(4) The cross sectional shape of shear wall, as a general rule, allowed in this design guideline is of I-shape. The same thickness of a wall panel as the column width, whose cross sectional shape is not of I-shape, would be allowed under the condition the column width should not be less than 40 cm. In this case the column depth might be recognized as the column width in the transverse direction. The minimum ratio of a wall thickness to a wall panel height is larger than that specified in the current standard, considering that the wall yields at its bottom end.

There is no requirements about the shape and the dimensions of an opening in a shear wall. But these are specified in the current standard, and vertically or horizontally long opening should be avoided to maintain the monolithic behavior and the flexural capacity of a wall.

(5) The nominal size of longitudinal reinforcement in a beam and a column should not be less than D19 to prevent the compression reinforcement from buckling. Assuming that the γ-ratio (the ratio of the amount of compression steels to the amount of tensile steels) is equal to 0 5, and the most severe combination of reinforcement and concrete strength, the tensile reinforcement of a beam could yield up to p_t of 3.3%. However, in order to prevent spalling of cover concrete or a compression yield of reinforcement, the tensile reinforcement ratio in a beam, including the tensile reinforcement of a floor slab, should not be more than 2.5% Where p_t is given by the ratio of the tensile reinforcement area to the area of a beam width multiplied by an effective beam depth. An adequate lateral reinforcement same as required in special hinge region specified in Chapter 9 is recommended to be provided in the case that p_t is more than 2%

A large tensile reinforcement ratio might cause not only to a poor flexural ductility but also to a bond failure, so that the adequate reinforcement is required according to Chapter 6.

As for the column there is no limit on p_t, because of column is usually 1.0. However, the large tensile reinforcement ratio p_t would result in a bond splitting failure as in the case of a beam. Previous researches show that a bond splitting failure would likely occur in the case of p_t of more than 1%. Then p_t of a column with yield hinge would be designed to be less than 1%.

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CHAPTER 6 : DESIGN FOR SHEAR AND BOND

6.1 Scope

The provisions of Chapter 6 shall apply for design of members subjected to shear as follows:

(1) design to ensure shear strength of columns, beams and structural walls,
(2) design to ensure deformation capacity of yield hinge regions of the members subjected to shear, and
(3) design to prevent a bond splitting failure along the longitudinal reinforcement in columns and beams

[Commentary]

The provisions of this Chapter should apply for the shear design of the non-hinge regions of columns, beams and structural shear walls, and for the ductility design of their hinge regions. Design for the bond applies for columns and beams in the yield mechanism assuring design.

6.2 Design Method

6.2.1 Basic Principles

In the shear design, the reliable shear strength of all members shall be greater than the design shear in the yield mechanism assuring design, and the deformation capacity of planned yield hinge regions shall be greater than the assuring deformation of member. In columns and beams, the splitting bond strength of longitudinal reinforcement shall be greater than the bond stress associated with design actions in the yield mechanism assuring design.

6.2.2 Strength of Shear Reinforcement

The strength of shear reinforcement shall be the material strength for the reliable strength calculation

6.2.3 Structural Requirements

Lateral reinforcement shall follow the provisions in Chapter 9 in addition to the provisions of this chapter.

[Commentary]

The strength of shear reinforcement used for the shear design is determined by the material strength for the reliable strength calculation. It should be, however, not greater than 25 times the compressive strength of concrete σ_B. The equations to evaluate shear strength of members proposed hereinafter can give the calculated results in good agreement with the empirical results, when substituting the strength of reinforcing bars of 25 times σ_B for the material strength of shear reinforcement that is greater than 4,000kgf/cm^2 Within this guidelines, when the material strength of shear reinforcement exceeds 25 times σ_B, the strength of shear reinforcement used for the shear design shall be replaced by the value of 25 times σ_B

When using such a high strength steel, its bending performance should be examined in accordance with the JIS-Z 2248 (Test Method for Bending Performance of Metallic Material) to prevent a brittle failure at the bend corner. And sufficient extension length beyond a hook is required when the 135 degree hooked bar anchorage is used for anchorage of a high strength shear reinforcement Either the serial spiral or closed hoop worked by welding is recommended to develop the full capacity of a high strength material. In this case, the welding joint should be provided with greater strength than the specified yield strength of the base material

6 3 Shear Strength of Beams and Columns

6 3.1 Strength Equation

Reliable shear strength V_u of the beams and columns shall be calculated by Eq. (6 1). When $p_w\sigma_{wy}$ is greater than $\nu\sigma_B/2$, $p_w\sigma_{wy}$ shall be replaced by $\nu\sigma_B/2$

$$V_u = b\, j_t\, p_w\, \sigma_{wy} \cot\phi + \tan\theta\,(1\text{-}\beta)\, b\, D\, \nu\, \sigma_B/2 \qquad (6.1)$$

where

$$\tan\theta = \sqrt{(L/D)^2+1}\ \text{-}L/D \qquad (6.2)$$

$$\beta = \{(1+\cot^2\phi)p_w\sigma_{wy}\}/(\nu\sigma_B) \qquad (6.3)$$

b: width of the section;
D. overall depth of the section;
j_t: distance between the top and bottom bars;
L: clear span of the member;
σ_B: compressive strength of concrete;
σ_{wy}: strength of the shear reinforcement not greater than 25 σ_B;
p_w: shear reinforcement ratio;
ν : effectiveness factor for the compressive strength of concrete; and
ϕ : angle of the compressive strut in the truss mechanism.

6.3.2 Coefficients for Members without Planned Yield Hinges

Effectiveness factor for the compressive strength of concrete ν shall be replaced by ν_o given by Eq.(6.4) for the members without the planned yield hinges.

$$\nu_o = 0.7 - \sigma_B/2000 \qquad (\sigma_B \text{ in kgf / cm}^2) \qquad (6.4)$$

The value of cot ϕ shall be the minimum defined in Eqs (6.5) through (6 7)

$$\cot\phi = 2\ 0 \qquad (6\ 5)$$

$$\cot\phi = j_t / (D \tan\theta) \qquad (6\ 6)$$

$$\cot\phi = \sqrt{\nu\sigma_B/ (p_w\, \sigma_{wy}) - 1} \qquad (6\ 7)$$

[Commentary]

(1) Shear strength of columns and beams

The prediction for the shear design in this section is fundamentally based on the lower bound theory of plasticity [Refs. 6 1-6 4]. Superposition of the truss and arch actions is introduced in modeling of the shear resisting mechanism Assumed plastic condition are those; (1) the total diagonal compressive stress in concrete generated by the combined truss and arch actions reaches the stress at the yield point of concrete, and (2) the stress in shear reinforcement reaches the stress at the yield point of shear reinforcement

The effectiveness factor, ν_o, in Eq. 6 4 proposed by M P. Nielsen [Ref. 6.1] is used in determining the stress at the yield point of concrete The stress at the yield point of shear reinforcement is given by the material strength for the reliable strength calculation of members. However, it should be not greater than 25 σ_B, because the equations to calculate the shear strength described in this guidelines correspond well to the test results by using 25 times σ_B for the material strength of shear reinforcement for specimens whose shear reinforcement strength is greater than 25 times σ_B.

Only the balance between external and internal shears is considered. It indicates the assumption that the flexural reinforcement has sufficient strength to assure the truss and arch mechanisms.

The first term in right-hand side of Eq 6 1 represents the shear force carried by the truss mechanism as shown in Fig. C6 1, and the second term indicates that carried by the arch mechanism as shown in Fig. C6 2.

The shear force carried by the truss mechanism, V_t, assuming the yield of shear reinforcement is described by Eq. C6.1 (Refer to Fig C6.1)

$$V_t = b\, j_t\, p_w\sigma_{wy} \cot\phi \qquad (C6.1)$$

Concrete stress in the compression strut of analogous truss, ${}_c\sigma_t$, is given by Eq. C6.2 from the equilibrium condition of an infinitesimal stringer element shown in Fig.1.(a).

$${}_c\sigma_t = (1 + \cot^2\phi)\, p_w\sigma_{wy} \qquad (C6.2)$$

The difference between $\nu\sigma_B$ and ${}_c\sigma_t$, i e , $(\nu\sigma_B - {}_c\sigma_t)$, contributes to the arch mechanism when ${}_c\sigma_t$ is smaller than $\nu\sigma_B$ The difference of angle of concrete struts between the arch and truss mechanisms is ignored herein for simplification The shear force carried by the arch mechanism, V_a, is given by Eq C6.3 based on the lower bound of the theory of plasticity [Ref 6 1]

$$V_a = (\nu\sigma_B - {}_c\sigma_t) \tan\theta\, b\, (D/2) \qquad \text{(C6 3)}$$

where

$$\tan\theta = \frac{\sqrt{L^2 + D^2} - L}{D} = \sqrt{\left(\frac{L}{D}\right)^2 + 1} - \frac{L}{D} \qquad \text{(C6.4)}$$

The shear strength of member, V_u, is given by adding strengths in Eq. C6.1 and Eq C6.3.

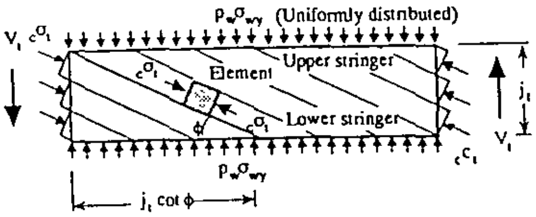
$$V_u = b\, j_t\, p_w\sigma_{wy} \cot\phi + (\nu\, \sigma_B - {}_c\sigma_t) \tan\theta\, (D/2) \qquad \text{(C6.5)}$$

Replacing $\{(1+\cot^2\phi)p_w\sigma_{wy}\}/\nu\sigma_B$ by β, Eq 6.1 is obtained. Angle ϕ is the angle of the concrete compression strut to the axis of member at the truss mechanism. The value of $\cot\phi$ should take the minimum given by Eqs 6.5, 6 6, and 6.7

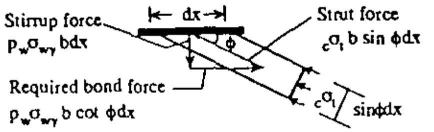
Equation 6.5 gives the allowable maximum value of $\cot\phi$ to assure appropriate aggregate interlocking along a diagonal crack [Ref. 6.5], and Eq 6.6 gives ϕ value to get maximum of V_u in Eq 6 1 Equation 6.7 is derived from the condition that $\nu\, \sigma_B$ equals ${}_c\sigma_t$ The Eqs. C6.6 and C6.7 are derived as follows.

$${}_c\sigma_t = (1+\cot^2\phi)\, p_w\sigma_{wy} = \nu\, \sigma_B \qquad \text{(C6.6)}$$

$$\cot\phi = \sqrt{\nu\sigma_B/p_w\sigma_{wy} - 1} \qquad \text{(C6 7)}$$



a) Analogous truss model



b) Equilibrium of an infinitesimal stringer element

Fig. C6.1 Truss mechanism.

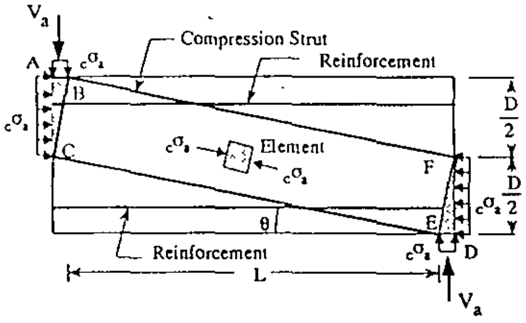


Fig. C6.2 Arch mechanism.

Equations in this section are established by using the value of $\cot\phi$ giving the maximum shear strength under the condition that $\cot\phi$ should not be greater than 2, and the concrete stress of the compression strut at the truss mechanism, ${}_c\sigma_t$, should not be greater than effective compressive strength of concrete, $\nu\, \sigma_B$. General characteristics of the design equations in this section are illustrated in Fig. C6.3'.

The limitation of shear reinforcement to the shear strength is given for the case that all shear forces are carried only by the truss mechanism with the angle ϕ of 45 degree ($\cot\phi$=1), and ${}_c\sigma_t$ equals $\nu\sigma_B$. When ${}_c\sigma_t$ equals $\nu_0\sigma_B$, Eq C6.7 is obtained Substituting Eq. C6.7 to Eq C6.1, and taking its differential by $p_w\sigma_{wy}$, the peak value $V_{t,max}$ is obtained as Eq. C6.8, where, $p_w\sigma_{wy}$ is $0.5\nu\sigma_B$ and $\cot\phi$ is unity

$$V_{t,max} = b\, j_t \nu\sigma_B/2 \qquad \text{(C6.8)}$$

The effectiveness factor of the compressive strength of concrete, ν_0, becomes smaller with increase of the compressive strength [Ref. 6.1]. Equation 6.4 takes this tendency into consideration

Within this guidelines, two methods of prediction for calculating the shear strength are proposed in the W.G on Shear Design (Task-committe organized for works for this chapter); the so-named A- and B-methods [Refs. 6 6-6 9]. Both the A- and B-methods are based upon the plastic theorem in the limit analysis, while are derived from the different concepts concerning empirical equations for shear design introduced in the previous sections. Through discussions within the Committee, the A-method is tentatively introduced as the prediction method proposed within this guidelines In the commentary herein, the B-method is introduced as well the A-method for a possible and wide use of the B-method

Both the A- and B-methods are based upon similar concepts with each other superposing the truss and arch mechanisms. The differences of the methods can be summarized in the values of $\tan\theta$, $\cot\phi$ and ν_0 as listed in Table C6.1.

TABLE C6.1 COEFFICIENTS IN THE A- AND B-METHODS

	A-method	B-method
$\tan\theta$	$\sqrt{(L/D)^2+1}-L/D$	$\sqrt{(2M/VD)^2+1}-2M/VD$
$\cot\phi$	$1.0\le\cot\varphi\le 2.0$, and smallest value among the following three 2.0 $j_t/(D\tan\theta)$ $\sqrt{\nu\sigma_B/(p_w\sigma_{wy})-1}$	1.0
ν_o	$0.7-\sigma_B/2000$	$(2M/VD+1)/4 \quad 0.5\le\nu_o\le 1.0$

M : bending moment at the critical section
V : shear at the critical section
D : dimension of the total section

In the A-method, the value of $\cot\phi$ that falls in the range of one and two is given on the condition that the truss mechanism associated with some amount of shear reinforcement could carry the maximum shear force While, in the B-method, the value of $\cot\phi$ is fixed to be 1.0 on the condition that the concrete stress of the compression strut in the truss mechanism associated with some amount of shear reinforcement takes the minimum stress

The shear strength predicted by the A-method is always greater than that by the B-method if the same stresses at the yield point of materials are used, and the moment distribution within the member is has anti-symmetric Based upon the lowest theorem, the A-method would estimate a real shear strength rather than the B-method

Both the A- and B-methods estimate greater shear strengths than those obtained from the test results when ν is assumed to be unity. Because the compressive strength of the cracked concrete might be smaller than that of the concrete without cracks, and concrete does not show an ideal elasto-plastic behavior, it is necessary to introduce the concept of reduction factor for the concrete strength.

In both methods, the shear force carried by the arch mechanism, in other words, the contribution of concrete to the shear strength, is varied associated with the amount of $p_w\sigma_{wy}$, while within the empirical equations for the shear strength introduced in the previous sections it is taken constant. This characteristic evidence that the shear force carried by the arch mechanism decreases in accordance with the increase of $p_w\sigma_{wy}$ is reported in the literature [Ref 6 2], and also be verified by the F.E.M. analysis [Refs 6.10 and 6.11]

In the A-method, ν takes the value of $(0.7-\sigma_B/2000)$, while in the B-method ν takes unity in general cases, which in some cases depends on the ratio of 2M/VD as listed in Table C6.1 in order to take the variation within the test results into consideration The value ν in the B-method takes the value within 0.5 and 1.0

General characteristics of the A-method are illustrated in Fig. C6.3' In the A-method, the $\cot\phi$ value is kept 2.0 until the $p_w\sigma_{wy}$ value reaches $0.2\nu_o\sigma_B$, and both the truss and arch mechanisms exist up to this point. Beyond this limiting point (point B in Fig C6.3'), the arch mechanism does not exist, and all shear forces should be carried only by the truss mechanism. The shear force carried by the truss mechanism can be increased beyond the point B up to its maximum value of $0.5bj_t\nu_o\sigma_B$, because of change of the angle of the compression concrete strut (ϕ=26.6 to 45 degree) While, in the B-method, the angle of ϕ is fixed to be 45 degree so that both the truss and arch mechanisms exist, and V_u versus $p_w\sigma_{wy}$ shows a linear relation until $p_w\sigma_{wy}$ reaches $0.5\nu_o\sigma_B$, and at the maximum point of $0.5bj_t\nu_o\sigma_B$ the shear force carried by the arch mechanism becomes zero The shear strength are different between in the A- and B-methods because they use the different ν_o values The shear strengths predicted by the A- and B-methods correspond well to each other without large difference within the range of $p_w\sigma_{wy}$ commonly used in the design.

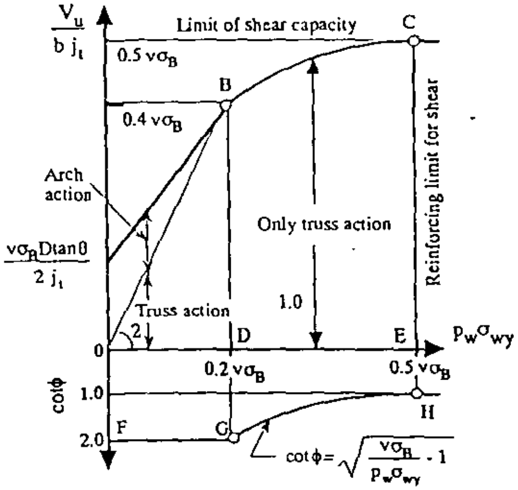


Fig. C6.3' General characteristics of design equation.

Under a conservative judgment, the A-method, which gives less prediction of shear strengths than the B-method, is adopted as the shear design equation in this section. Discussions necessary on the validity of both methods are summarized in the followings.

Both the A- and B-methods do not consider the effect of the axial forces. This is an issue to be examined in the future.

(2) Validity of the equation for the shear strength

Test results for the shear strength of columns and beams [Refs. 6.13–6.26] are referred for verification of the shear design equation adopted to this section. These test specimens are limited to those with deformed bars as flexural reinforcement, since the truss mechanism needs some bond strength between reinforcing bars and concrete. And also all specimen have not tensile axial forces (N≥0) and cross sectional area are larger than 400cm². Variables in those test specimens are listed in Table C6.2

TABLE C6.2 VARIABLES IN THE TEST SPECIMENS

Compressive strength of concrete σ_B	165-629 kgf/cm²
Tension reinforcement ratio.p_t	0.39-3.21 %
Shear reinforcement ratio.p_w	0-2.44%
Yield strength of shear reinforcement σ_{wy}	2530-14700 kgf/cm²
$p_w\sigma_{wy}$	0-191 kgf/cm²
Axial load level η_0=N/(bDσ_B)	0-0.732

Correlation between the test results and calculated ones using the A-method is plotted in Fig C6.4. Vertical and horizontal axes represent V_{max}/V_f and V_u/V_f, respectively. The value V_{max} is experimentally obtained the shear strength of test specimens, V_f is the shear force at the calculated flexural strength, and V_u is the calculated shear strength by the A-method. The value V_f is obtained based on the Bernoulli-Euler's assumption (the assumption that the plane section remains plane after bending) and using real strengths of steel and concrete. The reason why the axes as shown in Fig C6.4 are chosen is to confirm the fact that reported shear strength of the test specimens reaches some limiting strength, which is determined from the flexural capacity of the specimen [Ref. 6.17]. The specimen plotted in the zone between V_{max}/V_f less than 1.0 and V_u/V_f greater than 1.0 were reported to be failed in flexure. There are few specimens that have less strength than the calculated strength in the range of V_u/V_f less than 1.0. Among 77 specimens covering the range of variables listed in Table C6.3 whose V_{max}/V_f are less than 1.0 with shear reinforcement, the mean of V_{max}/V_u and its deviation are 1.33 and 18.5%, respectively. These values are obtained excluding specimens which have the V_{max}/V_u ratio less than 1.0 and those which are predicted to reveal bond failure in accordance with the section 6.5 in this chapter. As for specimens with high strength steel [Refs. 6.25 and 6.26], the mean of Vmax/Vu and its deviation are 1.41 and 17.9%, respectively.

TABLE C6.3 VARIABLES OF THE SPECIMENS FAILED IN SHEAR

Shear reinforcement ratio.p_w	0.12-1.13 %
Yield strength of shear reinforcement σ_{wy}	2550-14220 kgf/cm²
$p_w\sigma_{wy}$	3.16-159 kgf/cm²

The plastic theory used in this design guidelines does not consider the axial force effect. When an axial force is small, its effect is recognized experimentally, which is considerably significant for members without shear reinforcement, while the shear strengths of the specimens with some amount of shear reinforcement are recognized not to be affected significantly by axial forces. As shown in Fig. C6.4, the safety margin given to the shear strength determined by the shear design equation are almost constant with various amounts of axial force. Therefore, it is concluded that the effect of axial forces is not introduced within the equation.

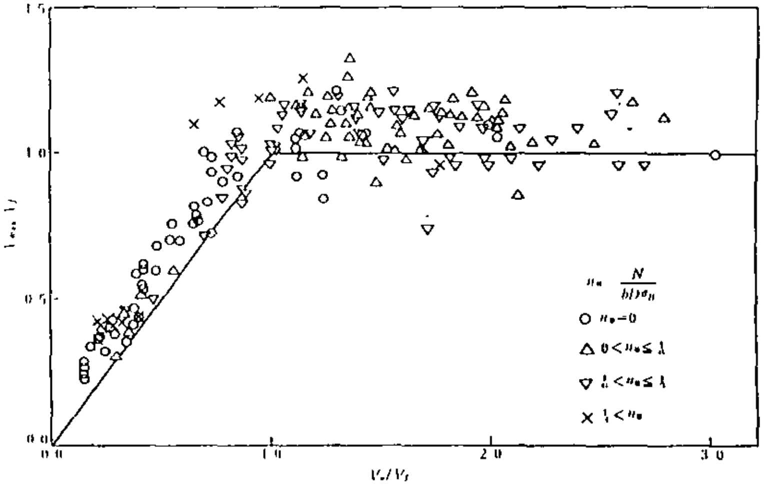


Fig. C6.5 Verification of the design equation in the A-Method.

(3) In the case of a member with solid circular section

The shear design method for members having solid circular cross section has not been established yet based on a plastic analysis, and few test data can be obtained. For the time being, the following method is recommended in this guidelines. Shear design for a member with solid circular section is performed as a member with square section of the same cross sectional area. The reduction factor of the shear reinforcement ratio of 0.785 ($\pi/4$) is prescribed, since the effect of shear reinforcement on the circular shape is reported to be less than that on the rectangular shape [Ref. 6.39].

6.3.3 Coefficients for Members with Planned Yield Hinges

(1) For the yield hinge region defined in Section 9.2 3, the effectiveness factor , v , shall be given by Eq (6.8) Value of cotϕ shall be the smallest value given by Eqs (6 6), (6.7) and (6 9). However, the value β in Eq (6 3) shall calculated using the value of cotϕ for a non-yield hinge region and the value of $p_w\sigma_{wy}$ for a yield hinge region of the member

$$\begin{aligned} v &= (1.0 - 15\,R_p)\,v_o && \text{for } 0 < R_p \le 0.05 \\ &= 0\,25\ v_o && \text{for } 0\,05 < R_p \end{aligned} \qquad (6\,8)$$

$$\begin{aligned} \cot\phi &= 2\,0 - 50\,R_p && \text{for } 0 < R_p \le 0.02 \\ &= 1.0 && \text{for } 0\,02 < R_p \end{aligned} \qquad (6\,9)$$

where R_p denotes the rotational angle at the yield hinge region associated with the assuring deformation of the member.

(2) Strength calculation of a region outside of the planned yield hinge regions shall use the effective factor given by Eq (6.8) The value of cotϕ shall be the smallest value given by Eqs (6.5) through (6 7) The value of β shall be that used for the yield hinge region.

[Commentary]

The deformation capacity of yield hinge is given by assuring both the curvature ductility at the critical section and the shear mechanism. The former one is assured by limit of axial force, preventing buckling of compression steel and appropriate confinement as provided in Chapters 5 and 9. In order to assure the shear mechanism, this design guideline gives the strength margin to concrete compression strut and changes the angle of truss mechanism according to the required deformation. The larger amount of lateral (confinement) reinforcement, required by maintaining either curvature ductility or shear mechanism, are actually arranged in members.

To prevent shear failure at yield hinge region, the effectiveness factor of compressive strength of concrete, v, and compression strut angle in truss mechanism, ϕ, are given as the function of required rotational angle at hinge region, R_p, by Eqs. 6.8 and 6.9. Figures C6.5 and C6 6 show the relationships of cotϕ and R_p, and of v and R_p, respectively. These relationships are based upon the concept that compression strut angle in truss mechanism, ϕ, increases up to 45 degree due to the loss of aggregate interlocking in post yield range , and final shear failure of member subjected to bending-shear forces would occur by crushing the concrete compression strut [Ref. 6.46]. In practical design, some strength margin against the design shear force of hinge region are indirectly given to shear reinforcement and concrete compression strut according to the required rotational angle, R_p

Design method for ductile members described in this chapter gives different amount of shear reinforcement for hinge region and outside hinge region in a member, respectively. As illustrated in Fig C6.11', the angle of concrete strut of truss mechanism , ϕ, changes gradually in a transition zone from hinge region to outside hinge region (Zone BCGF in Fig C6 11'). However, it should be noticed that maximum compressive stress in concrete would occur at point C in Fig C6 11' because higher lateral stress by shear reinforcement in hinge region and lower inclination of concrete strut in outside hinge region Therefore, in the calculation of β in Eq 6 3 (β is a coefficient indicating the level of compressive stress in concrete due to truss mechanism), $p_w\sigma_{wy}$ in hinge region and cotϕ in outside hinge region should be used The combinations of each coefficient to be used in the design are summarized in Table C6.4

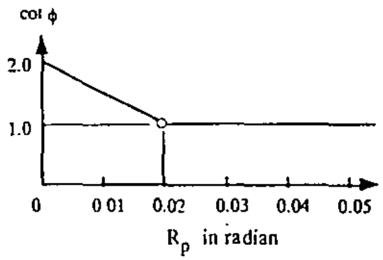


Fig. C6.10 Relationship between the guaranteed hinge rotation R_p and cotϕ

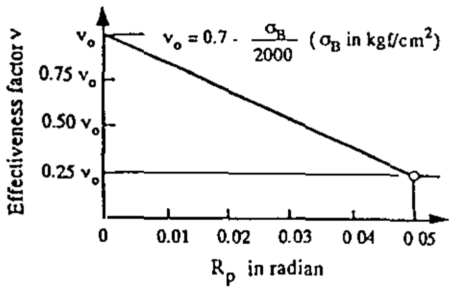


Fig. C6.11 Relationship between the guaranteed hinge rotation R_p and v.

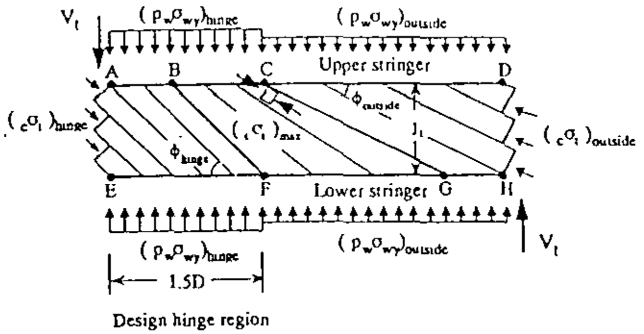


Fig. C6.11' Truss mechanism of a ductile member.

TABLE C6.4 VALUES OF v, $\cot\phi$ AND β USED*
IN THE DESIGN OF DUCTILE MEMBERS

Region	v	$\cot\phi$ in Eq 6 1	β
Hinge region	$0<R_p\leq 0\ 05$ $v=(1-15R_p)v_o$ $0\ 05<R_p$ $v=0.25v_o$	Smallest one of $\cot\phi=2-50R_p \quad R_p\leq 0\ 02$ $=1 \quad 0.02<R_p$ $\cot\phi=j_t/(D\tan\theta)$ $\cot\phi=\sqrt{v\sigma_B/(p_w\sigma_{wy})-1}$	$\beta=(\cot^2\phi+1)p_{wh}\sigma_{wy}/v\sigma_B$ where $\cot\phi$ is the smallest one of $\cot\phi=2$ $\cot\phi=j_t/(D\tan\theta)$ $\cot\phi=\sqrt{v\sigma_B/(p_w\sigma_{wy})-1}$
Outside hinge region	ditto	Smallest one of $\cot\phi=2$ $\cot\phi=j_t/(D\tan\theta)$ $\cot\phi=\sqrt{v\sigma_B/(p_w\sigma_{wy})-1}$	ditto

p_w : shear reinforcement ratio in the hinge region

Note · Different amount of shear reinforcement are arranged in the hinge region and in the region outside the hinge region

Correlation between test data [Refs. 6 29–6 34] and predicted results are shown in Fig. C6.12. Specimens which showed bond splitting failures are excluded Vertical axis indicates experimentally observed member rotation angle at ultimate state and horizontal axis indicates theoretically predicted available member rotation angle. Ultimate angle of specimens are defined as the angle at 80% strength after peak load on experimentally observed load-deformation diagram. Calculated member rotation angle is obtained by adding the contribution of predicted hinge rotation R_p and the member rotation angle at yielding defined in [Ref. 6 49]. Ductility capacity might be predicted by the method described in this section except for members subjected to extremely large axial force. Equations 6 8 and 6.9 assume the rotation at yield hinge, so that length of hinge region and deformation other than of hinge region are necessary to get member's drift. From the point of conservative judgement, the rotation at yield hinge, R_p, could be used for member's drift.

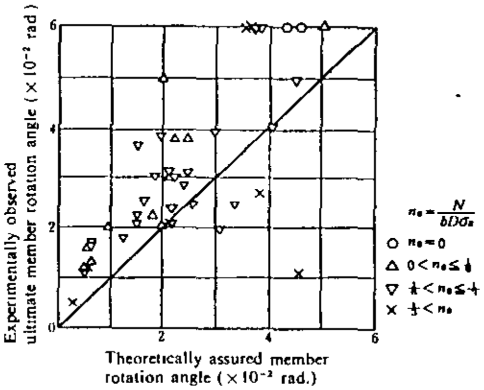


Fig. C6.12 Verification results of ductility.

On the other hand, shear design of ductile member can be conducted by using v and $\cot\phi$ given for hinge region in Table C6 4 regardless of hinge and outside hinge region. This gives uniform shear reinforcement across the member. In this case, the compressive stress in concrete due to truss mechanism becomes smaller and compressive stress in concrete due to arch mechanism becomes higher than those in the design method according to Table C6.4, and it results smaller amount of shear reinforcement in hinge region and larger amount of shear reinforcement in outside hinge region.

6.3 4 Inclined Shear Reinforcement

When the inclined shear reinforcement is used, the shear given by Eq.(6 10) may be added to the shear strength

$$V_x = A_x \sigma_{xy} \sin\theta_x \qquad (6\ 10)$$

where

A_x : sectional area of inclined reinforcement;

σ_{xy}. yield strength of inclined reinforcement; and

θ_x: angle between inclined reinforcement and member axis.

Area A_x may include both tension and compression reinforcement if the inclined reinforcement is placed diagonally across the member, otherwise, the area of inclined reinforcement in tension shall be used

[Commentary]

Inclined shear reinforcement in yield hinge region shown in Fig 6 9(a) have been used as a bent reinforcement, and they can carry the shear force by the shear component of their tension forces. Shear force given by Eq 6 10 would be added to the shear strength given by Eq. 6 1. Such a inclined shear reinforcement is effective on the case that the inelastic tensile strain of longitudinal reinforcement is accumulated due to many cyclic bending action, and then sliding shear failure occurs due to full crack opening at beam critical section. The inclined angle to member axis, however, should be ranged from 30 to 45 degree. Safety check for bearing stress of concrete inside the bent corner, and the contribution of inclined reinforcement on the flexural strength at yield hinge should be considered as well as providing enough embedment length.

The so-called X-shaped reinforcement arranged diagonally across the members as shown in Fig.6.9(b) are known to be effective on shear strength, experimentally and theoretically [Refs. 6.35–6 38]. Steel areas of both tension and compression might be countable as the steel areas used in Eq. 6 10 This X-shaped reinforcement could contribute to flexural strength as well as to shear strength. And X-shaped reinforcement does not need bond mechanism for its shear resisting mechanism, then check for bond strength might be done to the residual parallel flexural reinforcements. It is possible to avoid bond splitting failure using this X-shaped reinforcement. The X- shaped reinforcement could change the failure mode of the member with itself from shear failure including bond splitting failure to flexure failure under any conditions, and make even the members subjected to large bending moment and shear very ductile Details are described in Refs. 6 35 to 6 38

The members with inclined reinforcement should be designed as stirrups or hoops carry a part of shear force. The ratio of shear force carried by stirrup or hoops should be determined carefully according to previous test data and research, and shear force carried by inclined reinforcement should not be overestimated. In the New Zealand code [Ref. 6 48], it is recommended that stirrups or hoops should carry at least one-third of total shear force.

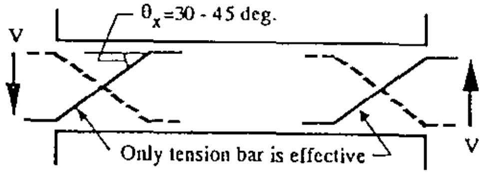


Fig. C6.14 X-shaped reinforcement in the hinge regions.

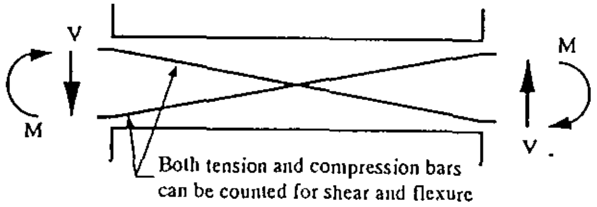


Fig. C6.15 X-shaped reinforcement placed diagonally across a member.

6.3.5 Minimum Amount of Lateral Reinforcement

Minimum shear reinforcement ratio shall be 0.2 percent in all beams and columns.

6.4 Shear Strength of Walls

6.4.1 Strength Equation

Reliable shear strength V_u at each story of a wall shall be calculated by Eq (6.11). When $p_s\sigma_{sy}$ is greater than $\nu\sigma_B/2$, $p_s\sigma_{sy}$ shall be $\nu\sigma_B/2$

$$V_u = t_w\, l_{wb}\, p_s\, \sigma_{sy} \cot\phi + \tan\theta\, (1 - \beta)\, t_w\, l_{wa}\, \nu\, \sigma_B/2 \qquad (6.11)$$

where

$$\tan\theta=[\sqrt{(h_w/l_{wa})^2+1} - h_w/l_{wa}] \qquad (6.12)$$

$$\beta=(1+\cot^2\phi)p_s\sigma_{sy} / (\nu\sigma_B) \qquad (6\ 13)$$

t_w: thickness of the wall panel;
h_w: height of the wall, which may be taken as the inter-story height,
l_{wb}: equivalent width of the wall panel in the truss mechanism (see Clause 6.4.2),
l_{wa}: equivalent width of the wall panel in the arch mechanism (see Clause 6.4.2),
σ_B: compressive strength of concrete,
σ_{sy}: strength of shear reinforcement within the wall panel not greater than 4000 kgf/cm^2;
p_s: shear reinforcement ratio within the wall panel;
ν: effectiveness factor for the compressive strength of concrete, and
ϕ: angle of the compressive the strut in truss mechanism, and $\cot\phi = 1.0$

[Commentary]

(1) Planned yield hinge mechanism

The provisions in this section should apply for the design of shear wall that carries lateral force. Failure mode of shear wall should be flexure with yield hinge at the bottom end, and the design of shear wall should follow the next provisions to prevent shear failure. Failure mode of lifting-up of a foundation might be allowed if shear wall has enough strength margin against shear failure.

The yielding mode of lifting-up of a foundation is thought to be also ductile for a shear wall as well as flexural yielding of a wall. Flexural yielding of foundation beams, instead of flexural yielding at the bottom end of the shear wall, would be desirable if possible, because there would be little damage on shear wall. Of course, ductility of foundation beam and foundation itself are required instead of that of shear wall, especially soil and piles in compression side should be designed carefully to assure enough strength and ductility. Shear force of wall with pile foundation has a possibility to become larger than that calculated at assurance design stage due to the undetermined pulling out strength of piles. In the case that lifting-up mechanism carries a large part of the horizontal shear capacity, there is a room to be discussed about the design shear coefficient, if the design shear coefficient is the same as that in the case of flexural yield, because lifting-up mechanism dissipate few energy. As for lifting-up mechanism, there are many problems to be made clear to get design criteria.

(2) Design shear

Design shear would be obtained, according to Chapter 4, based on the shear force at yield mechanism with the overstrength of yield hinges and magnified by the dynamic amplification factor. In calculation of design shear force for shear wall, following points should be taken care adding on the "Commentary" in Chapter 4.

i) Static shear is affected by not only the overstrength of shear wall, boundary beams and transverse beams, but also the overstrength of beams in other part of a structure. It is necessary to analyze the total structure considering shear transfer through floor system, i.e., it is not enough to analyze only a part including shear wall.

ii) When lifting-up mechanism is assured, the upper strength of foundation and foundation beams directly affect on the strength of this mechanism, however, the overstrength of pulling out of piles could not be estimated exactly, then it is required to set enough safety margin to their strength. At the present time, it is conservative and desirable to designate the force at flexural yielding of shear wall or the strength at tensile yielding of vertical reinforcement in the piles as the design shear.

iii) The additional story shear force due to the dynamic effect has a tendency to be carried mostly by shear wall, and then it is conservative and desirable to assume all additional story shear would be carried by shear wall. Vertical and horizontal reinforcement in a shear wall would be arranged basically equally, then adequate reinforcing arrangement is necessary before the calculation of design shear force for the assurance design, because flexural strength of wall is also affected by vertical reinforcement. The design shear force for shear wall (except for cantilever type wall) is not proportional to flexural strength, so that a larger amount of vertical reinforcement than that required in the design does not matter at many cases. But in the severe design case for shear, it is necessary to assure reasonable reinforcing arrangement according to design demand. If strength of shear wall would be short, it is desirable to make wall panel thicker or to use inclined reinforcement instead of increasing shear reinforcement in wall panel.

(3) Equation for shear strength

Equation for shear strength of shear wall is derived based on the same concept [Refs. 6.50, 6.51], plastic theory, as used for shear strength of beams and columns, where shear force is transferred by truss and arch mechanisms, and the strength of vertical steels is assumed to be infinite. Shear design of shear wall might be done by each story. The differences from the equations for beams and columns are following:

i) Strength of shear reinforcement should not be greater than 4,000kgf/cm^2, because there are no test data using high strength shear reinforcement for verification of this equation.

ii) Angle in truss mechanism takes better one which is determined in A-method ($\cot\phi \leq 2.0$) or B-method ($\cot\phi = 1.0$) to get a good correlation with test data.

iii) Effectiveness factor for compressive strength of concrete that assures deformation capacity is newly given considering test data.

iv) Area of boundary column is taken into account as the equivalent length of wall in arch mechanism.

v) The method assuring the shear transfer between stories is provided in order to conduct shear design of wall by each story.

Ninety-nine shear wall test specimens in eight test series satisfying next conditions are selected for verification [Refs. 6.52–6.65]:

(1) cyclic loading test;
(2) test in series with some particular parameters;
(3) test specimens with boundary columns, and
(4) test specimens with more than one-third of full scale

In those tests, compressive strength of concrete and yield strength of reinforcing steel were ranged from 200 to 400 kgf/cm^2 and 3000 to 4500 kgf/cm^2, respectively Test specimens are summarized in Table C6.5 As for the angle, ϕ, in truss mechanism, B-method, $\cot\phi=1\ 0$ constantly, could predict test data better than A-method, $\cot\phi=1\ 0\sim2\ 0$ Then $\cot\phi=1.0$ is adopted in shear design of a shear wall.

TABLE C6.5 SELECTED SPECIMENS FOR VERIFICATION

Test series	Reference	Number of specimen	Characteristics and parameters	Mark
1	6.52, 6.53	5	Lateral confinement of boundary column	○
2	6 54, 6 55	6	Variation of shear span ratio and wall thickness	·
	6 56	4	High axial load	●
3	6 57	34	Heavy wall reinforcement ratio	□
4	6 58	16	Axial load level and Shear span ratio	■
5	6 59, 6.60	5	Large shear span ratio	△
6	6.61	7	Small shear span ratio	+
7	6 62, 6.63, 6 64	20	Wall reinforcement ratio	▲
8	6.65	2	Shear span ratio	⊕

6.4.2 Equivalent Wall Widths

Calculation of the shear strength for a wall can use the equivalent wall widths including the effective length due to confinement by the boundary column as defined in Eqs (6.14) and (6.15).

$$l_{wa}=l'_w+D_c+\Delta l_{wa} \qquad (6\ 14)$$

$$l_{wb}=l'_w+D_c+\Delta l_{wb} \qquad (6.15)$$

where

l'_w: clear span of the wall panel;

D_c: width of the boundary column,

Δl_{wa}: increment of the wall width given by Eq (6 16), and

Δl_{wb}: increment of the wall width given by Eq (6 17)

$$\begin{aligned}\Delta l_{wa}&=A_{ce}/t_w && \text{for} \quad A_{ce}\le t_wD_c\\ &=(D_c+\sqrt{A_{ce}D_c/t_w})/2 && \text{for} \quad A_{ce}>t_wD_c\end{aligned} \qquad (6.16)$$

$$\begin{aligned}\Delta l_{wb}&=A_{ce}/t_w && \text{for} \quad A_{ce}\le t_wD_c\\ &= D_c && \text{for} \quad A_{ce}>t_wD_c\end{aligned} \qquad (6.17)$$

where

A_{ce}: effective area of the boundary column given by Eq (6 18) not greater than $3t_w\,D_c$.

$$A_{ce} = A_c - N_{cc} / \sigma_B \qquad (6\ 18)$$

where

A_c: sectional area of the compressive side boundary column; and

N_{cc}: axial load of the compressive side boundary column in the yield mechanism assuring design.

[Commentary]

It is made sure by also experiment that the area of boundary columns contribute the shear strength of a wall. Equation used in the current ultimate strength design replaces the total wall area including area of boundary column by equivalent rectangular area, and could predict test results well. But the contribution of boundary column to shear strength of wall could not be explained reasonably or quantitatively. Shear design equation adopted in this design guideline is derived from the research that evaluates the contribution of boundary column to the strength of wall theoretically based on equilibrium condition, and is simplified for a practical design use.

There is a research [Ref. 6.66] in which a virtual increment of wall width due to a existing of boundary column, where the angle of arch mechanism changes and shear strength of wall increases, can be estimated roughly by the equilibrium condition with flexural strength of boundary column. As shown in Fig. C6 16, a width of wall for arch mechanism extended from boundary column (Δl_c : the distance from the center of boundary column) is given by Eq. C6.13 assuming that the flexural strength of boundary column, M_{cu}, is equal to the moment about centroid of column at it's bottom produced by diagonal compression force within a width of Δl_c.

$$\Delta l_c=\frac{\sqrt{\dfrac{2M_{cu}}{\nu\sigma_B t_w(1-\beta)}}}{\cos\beta} \qquad \text{(Eq. C6.13)}$$

Flexural strength of boundary column is simply obtained as Eq. C6.14 ignoring the axial force effects.

$$M_{cu} = 0\ 8 p_t \sigma_y A_{cc} D_c \qquad \text{(Eq C6 14)}$$

where A_{cc}, p_t , σ_y designate an effective area of the boundary column given by Eq. 6.17 (excluding concrete area resisting to column axial force), the tensile steel ratio to A_{cc}, and the tensile strength of steel bars, respectively.

Statistics on another unknown values are assumed to satisfy Eq. C6.15·

$$\frac{1.6 p_t \sigma_y}{\nu \sigma_B (1-\beta) \cos^2\theta} \doteq \frac{1}{4} \qquad \text{(Eq C6 15)}$$

Equation C6 16 will be obtained

$$\Delta l_c = \frac{\sqrt{\frac{A_{cc} D_c}{l_w}}}{2} \qquad \text{(Eq C6 16)}$$

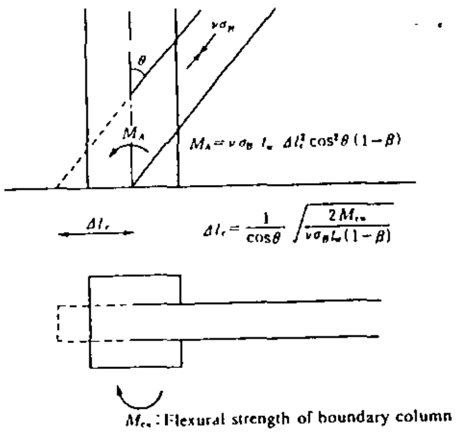


Fig. C6.16 Increment of the wall width due to confinement of a boundary column.

This simplification is considered to get $\Delta l_c = D_c/2$ at $A_{cc} = t_w D_c$. Equation C6 16 could give conservative results because the contribution of axial force to column flexural strength is ignored in Eq. C6.14. In tension side, the effect of boundary column should not be considered, and the effective area for arch mechanism should be up to the end of boundary column and equations 6 14 and 6 15 are obtained. Here, area of A_{cc} is limited up to $3t_w D_c$ considering many combination of wall thickness and shape of boundary column And concrete area required for resisting compressive force , N_{cc}, induced to boundary column due to wall axial load (dead load+live load) and bending moment (earthquake load) acting at the top of wall in that story should not be included in A_{cc}. The compressive force, N_{cc}, adding half of wall axial load , $N_w/2$, to axial loads , M_T/l_w , due to bending moment is described as Eq C6 17. The value, N_{cc}, obtained from Eq C6 17 is not always conservative

$$N_{cc} = N_w/2 + M_T/l_w \qquad \text{(Eq C6 17)}$$

where

N_w : axial load for the yield mechanism assuring design;

M_t : assuring design moment at the top of a wall in each story; and

l_w · distance between boundary columns, and 0 7 l_w gives a conservative result

Figure C6.17 shows the correlation between shear strength predicted by the design equation in this section and test results Y axis indicates the experimentally obtained maximum strength normalized by theoretical flexural strength of wall. X axis indicates predicted shear strength and also normalized by theoretical flexural strength Almost all test results are higher than predicted results, except three specimens (solid triangles). One of these three specimens was reported to failed at its loading beam. Other two specimens would be seemed to have the same problems. In the calculation of flexural strength , the flexural strength are calculated on the distance of on-center in forces defined by rectangular concrete stress block in compression side and yielding of all tensile steels in boundary column and vertical steels in wall panel For simplification, compressive concrete stress block is assumed to be within a boundary column, and the effect of compression steel is ignored.

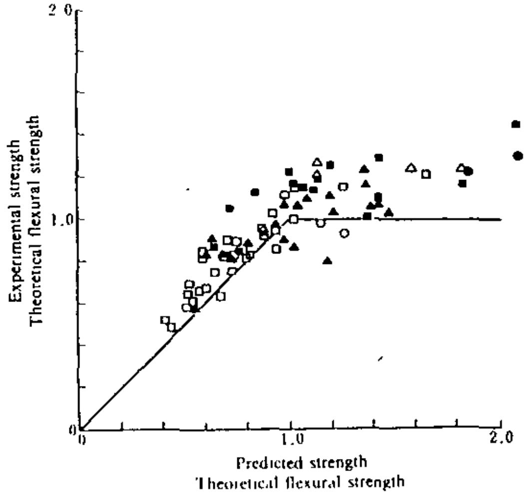


Fig. C6.17 Verification of the design equation for a non-hinge region.

6.4 3 Coefficient for Non-yield Hinge Regions

Calculation of the shear strength in non-yield hinge regions, defined in Clause 9 2 3, shall use the effectiveness factor for compressive strength of concrete given by Eq.(6 4)

[Commentary]

Effectiveness factor, v, for compressive strength of concrete would be given by the same equation used as beams and columns As same in the design for beams and column, it is good to reduce the effectiveness factor in order to get good predictions in the case of using high strength concrete

6.4 4 Coefficients for Yield Hinge Regions

Calculation of the shear strength in yield hinge regions shall use the effectiveness factor v for compressive strength of concrete given in Eq (6 19)

$$v = \begin{cases} v_0 & \text{for } R_u < 0\,005 \\ (1.2\text{-}40R_u)\, v_0 & \text{for } 0.005 \le R_u < 0\,02 \\ 0\,4 v_0 & \text{for } 0\,02 \le R_u \end{cases} \qquad (6\ 19)$$

where R_u denotes the assuring rotational angle of the wall.

[Commentary]

Effectiveness factor for compressive strength of concrete in yield hinge region specified in clause 9 2.3 is given differently from in non-hinge region, in order to assure the deformation capacity in hinge region more than required deformation capacity. The procedure to determine the effectiveness factor for compressive strength of concrete is described below.

Effectiveness factor of concrete strength, v_m, necessary to get the same calculated shear strength based upon inelastic theory as the flexural strength is discussed about the relationship with deformation capacity (ultimate deformation) That is, the concrete stress due to truss and arch mechanism at theoretical flexural strength of walls , σ_c , is theoretically calculated and then v_m is obtained by $v_m=\sigma_c/\sigma_B$. The relation between the ratio of v_m to v_0 (effectiveness factor of concrete compressive strength used for non-hinge region) and ultimate drift of test specimens, R_u, is shown in Fig. C6.19. Good deformation capacity is expected in the range of small of v_m/v_0, even if the plotted points are scattering The boundary value could be set as shown in Fig. C6 19. The expected deformation capacity can be assured by performing shear design by using the lowest effectiveness factor corresponding to that deformation.

For the verification, 49 specimens among 99 specimens listed in Table C 6 5 were selected, which showed ultimate drift angle of more than 1% Ultimate deformation is defined as the point at 80% of maximum strength on their envelope of load-deformation diagram When ultimate deformation is determined under cyclic loading path within the same drift, the envelope of load-deformation diagram extrapolated by connecting the maximum strength point to the drift of 1/50 are assumed. As for the test specimens modeling multi story shear wall, the first story drift is referred. The relation between the ratio of v_m to v_0 ,effectiveness factor of concrete compressive strength used in the discussion of shear strength, and ultimate drift, R_u, is shown in Fig C6.19 Good deformation capacity is expected in the range of small of v_m/v_0, even if the plotted points are scattering. The boundary value could be set as shown in Fig C6.19. The expected deformation capacity can be assured by performing shear design by using the lowest effective factor corresponding to that deformation.

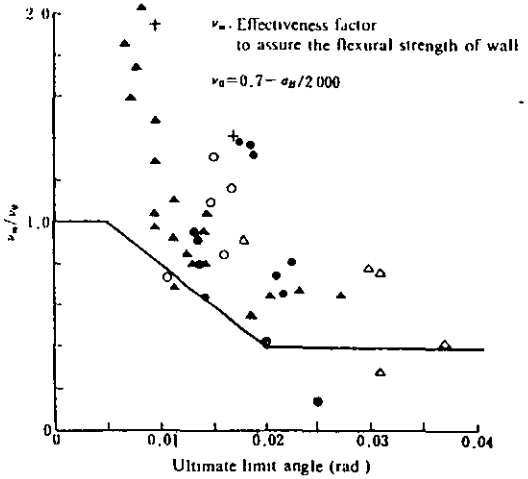


Fig. C6.19 Effectiveness factor of concrete, v_m , and deformation capacity.

In the design of shear wall in a wall-frame structure, the point is how to assure the strength and ductility in lower stories (hinge regions). Because the required shear strength in the upper story (non hinge region) may be easily assured. The current design method does not distinguish the difference between hinge and non-hinge region, however, in this design guideline, design for hinge regions is dominant. In hinge region, effectiveness factor (v=0.7v_0) is used for design in order to assure a deformation capacity (R_u=1/75), and then shear strength (potential shear strength) in hinge region calculated by using the reduced effectiveness factor (0.7v_0) against design force is higher than that calculated in the discussion of shear strength, where v is equal to v_0 The ratio of strength calculated by using v=0.7v_0 to test results is shown in Fig. C6 20 Test results always shows higher values than calculated strength. This means hinge region has enough safety margin to shear strength

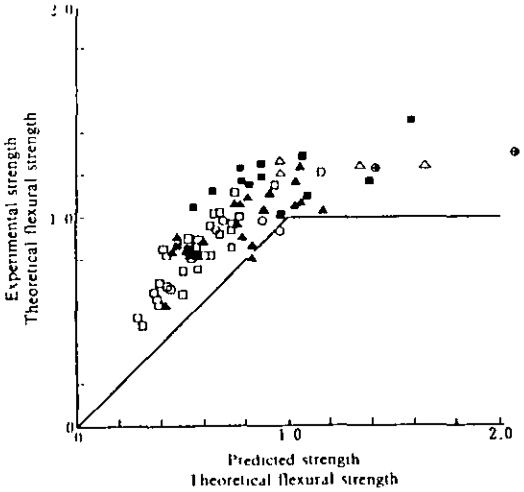


Fig. C6.20 Predicted shear strength using the reduced effectiveness factor (ν=0.7ν_0) for a hinge region.

ACI318-83 code specifies the upper limit of shear strength of wall to be $10\sqrt{f'_c}bd$ (psi), that is equal to $0.174f'_cbd=0.217f'_cb_wl_w$ ($d=0.8l_w$) at concrete strength f'_c= 240kgf/cm^2. And there is a research which recommends that nominal shear strength is less than $0\ 25f'_ct_wl_w$ (l_w.on-center distance of boundary columns) in order to assure the ductility after flexural yield. These upper bound limit are determined from experimental data.

The upper bound limit strength of shear wall calculated by the strength equation described in this section could be defined as the maximum strength at the time when the diagonal compressive stress of concrete in truss mechanism reaches the effective compressive strength of concrete (all of shear is carried by truss mechanism) That is, increasing the amount of shear reinforcement makes force carried by arch mechanism decreasing to zero (β=1), and maximum force carried by truss mechanism becomes $t_wJ_{wb}\nu\sigma_B/2$. When σ_B is equal to 240kgf/cm^2, this limit strength is $0.29\sigma_Bt_wl_{wb}$ for non-hinge region and $0.20\sigma_Bt_wl_{wb}$ for hinge region, and these correspond to the recommendation in ACI code.

6 4 5 Inclined Shear Reinforcement

When inclined shear reinforcement is used in a wall panel, the shear V_{wx} given by Eq (6.20) may be added to the shear strength of Eq (6 11):

$$V_{wx} = A_{wx}\ \sigma_{wxy}\ \sin\theta_{xw} \qquad (6\ 20)$$

where

A_{wx} : cross sectional area of the inclined reinforcement,

σ_{wxy}' yield strength of the inclined reinforcement; and

θ_{wx}: angle between the inclined reinforcement and wall axis.

[Commentary]

In Eq. 6.20, both tension and compression side wall reinforcement may be take into account. In some experiments, theoretically calculated shear strength of walls using Eq. 6.20 has higher safety margin to experimentally observed strength than that of walls with shear reinforcement normally arranged, vertically and horizontally. Therefore, it is not overestimated that inclined reinforcement, both tension and compression side, are effective on shear strength. There is some doubts about the quantity of effectiveness for inclined reinforcement, however, this is good for ductility as well as strength, then all area of inclined reinforcement might be countable to the shear strength. There are two methods of the arrangement of inclined reinforcement, one is distributed method like as normal arrangement of steel in a wall, and the other is concentrated method like as diagonal reinforcement. Both methods are acceptable, and have no problems. Steel area used in calculation, however, should be appropriately defined as horizontal force components of inclined reinforcement corresponds to increment of shear strength The wall specimen with concentrated inclined reinforcement with hoops in wall panel, whose shape is just like as a wall panel with diagonal column section, escapes an evident compression failure at the wall panel under large deformation situation (R=1/50) and small shear-span ratio (M/VD=0.5), and shows extremely excellent deformation capacity. This detail of inclined reinforcement is desirable for hinge region subjected to large shear force.

6 4 6 Transfer of Shear between Stories

Shear carried by the arch mechanism of a story, except for a part of it transferred directly to the immediately lower story by the arch action, shall be resisted by the tensile action in beams or by the truss mechanism in the immediately lower story

[Commentary]

Multi-story shear wall could be designed by each story by story in this design procedure, shear force in a story should be transferred to the immediately lower story satisfying the equilibrium of forces. Shear carried by arch mechanism of a story, except for the part transferred directly to the immediately lower story by arch action, should be resisted by tension in beams or by truss mechanism in the immediately lower story Practical design procedure is as following (see Fig. C6 21), where dashed symbol denotes variables on the upper story, and simple symbol on the lower story, for example, V' indicates design shear for upper story wall and V indicates design shear for lower story wall.

Shear carried by arch mechanism in lower story, V_a:

$$V_a=\tan\theta(1-\beta)t_w l_{wa}\nu\sigma_B/2 \qquad \text{(Eq. C6.18)}$$

and the part of shear transferred directly from the upper story to the lower story by arch action, V_{al}, is generally presented by Eq. C6.19 (transferred through overlapping area of arch struts in upper and lower stories).

$$V_{al} = V_a(1-\lambda\tan\theta - \lambda'\tan\theta')/(1-\lambda\tan\theta) \qquad \text{(Eq. C6.19)}$$

where $\lambda=h_w/l_{wa}$ and $\lambda'=h'_w/l'_{wa}$. If h_w is equal to h'_w and l_{wa} is equal to l'_{wa}, Eq C6 20 will be derived.

$$V_{al} = V_a\tan\theta/(\tan\theta + \lambda) \qquad \text{(Eq. C6.20)}$$

If the story height is different, it is safety judgement to use the greater story height.

Equation C6.21 shows shear carried by truss mechanism in the upper story

$$V'_t = \cot\phi t'_w l'_{wb} p'_s \sigma'_{sy} \qquad \text{(Eq. C6.21)}$$

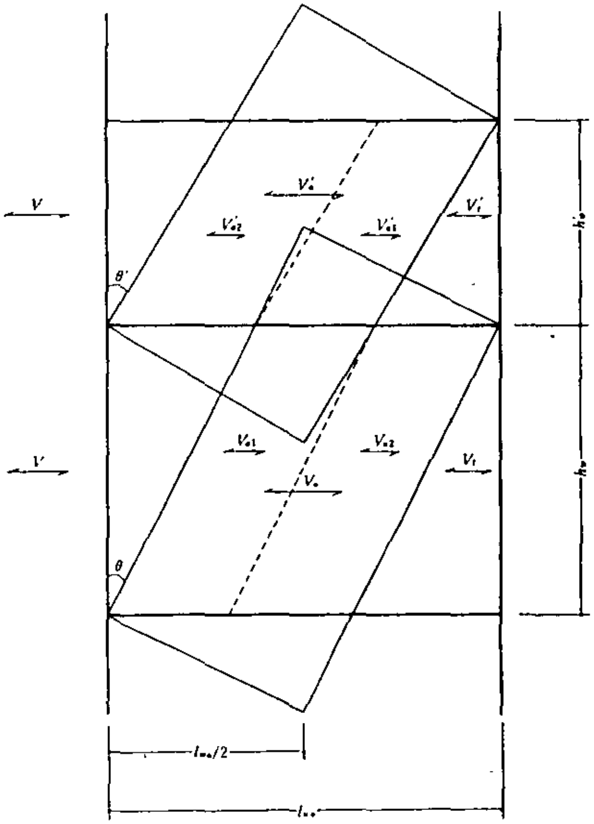
Exceeded shear that should be resisted by tension in beam is obtained by Eq. C6.22.

$$\Delta V = V' - (V'_t + V_{al}) \qquad \text{(Eq. C6.22)}$$

Tensile strength of beam, $T(=a_g\sigma_y$, a_g: total area of axial steels, σ_y:strength of steel), should be greater than or equal to ΔV

$$T \geq \Delta V \qquad \text{(Eq. C6.23)}$$

This procedure is assumed that the value of β in both lower and upper stories are always the same, and this assumption would give conservative results, because β of the lower story is always small. The reason why strength equation based on $\cot\phi=1.0$ gives good prediction to test results is thought that there are few test specimens which is good to verify the case of large value of $\cot\phi$ The angle in truss mechanism should be discussed from now, because the assumption of $\cot\phi$ of less than 2.0 at strength equation does not always give an unsafe result



V : design shear
V_u : shear strength
V_a : shear provided by the arch mechanism
V_t : shear provided by the truss mechanism
V_{a1} : shear transfered directly to the lower story through the arch strut
V_{a2} : $V_a - V_{a1}$

Fig. C6.21 Interstory shear transfer.

6.4 7 Minimum Amount of Shear Reinforcement

Shear reinforcement ratio in a wall panel shall be not less than 0.0025. Shear reinforcement ratio of the boundary columns shall be not less than 0 003 in a yield hinge region

[Commentary]

(1) Confinement of boundary column

Bottom end of boundary column in shear wall designed to yield in flexure, no matter how tall wall is, is subjected to large axial and shear forces This design guideline specifies the details of steel arrangement of boundary column according to its axial stress level assuming that the axial force due to bending moment at flexural yield strength of wall is resisted by the confined boundary column. But required amount of the confinement also depends on design shear level of wall, required deformation, and required seismic performance as a column in another direction. There is no reasonable way to take all these factors at present time.

Confinement at only the bottom part of boundary column is good, and the amount of steel in this part is very small to that used in the total building, then it is desirable to reinforce the part subjected to large axial force as much as possible It is recommended, not based on theory but on experiment, that volumetric ratio of lateral reinforcement to confinement zone is roughly more than 0.6%, 1.2%, and 1.8% for the ratio of axial force to compressive strength of less than 0 3, from 0 3 to 0.6, and more than 0.6, respectively The effect of confinement depends on a spacing of lateral reinforcement, then it is recommended its spacing of less than 30cm and less than 20cm for the part under high axial force [see Section 9.3].

(2) Structural requirements

Wall thickness should be more than 15cm and more than 1/30 of clear height of wall panel Shear reinforcement ratio in wall panel should be more than 0 0025 in each orthogonal direction. Shear reinforcement should be arranged in double in case of wall thickness of more than 20cm, and its nominal size should be more than D10 (10 mm in diameter deformed bar), and its spacing in front area should be less than 30cm, but in case of alternative arrangement of steel its spacing in one side is less than 45cm.

(3) Opening position, strength of wall with opening and reinforcement for opening

Having a opening in wall at non-hinge region has no problems if enough shear strength is provided. Even at hinge region, wall with openings, which locate at other than critical part and are enough reinforced to prevent shear failure, could behave like as a solid wall. Shear strength of wall with opening can not be easily estimated because it relates to the reinforcing method for opening However, the strength calculated by "AIJ Standard for Structural Calculation of Reinforced Concrete Structure," in which strength reduction factor concerning on opening ratio, area of opening to are of wall panel is introduced, will predict safely When the strength equation of this section applies to the wall with opening assuming that the wall consists of two walls divided at the opening, height of wall in the equation is very difficult to be fixed, that is clear height of opening gives unsafe prediction and clear height of wall panel gives too much safety prediction There is no simple and good way for this problem.

As for reinforcement arrangement around the opening, there are some special effective details such as X-shaped reinforcement in walls aside to opening, but these method is verified by only experiment and has not yet generally established as design method

6 5 Design for Bond

6 5 1 Design Bond Stress

Design bond stress shall be calculated by Eq.(6.21). For a member without planned yield hinges or with an planned yield hinge at the one end, the design bond stress can be given by the smaller value between those calculated by Eqs.(6.21) and (6.22).

$$\tau_f = \frac{d_b \Delta\sigma}{4\,(L-d)} \tag{6.21}$$

$$\tau_t = \frac{b\, p_{wt}\, \sigma_{wy} \cot\phi}{\Sigma\psi} \tag{6.22}$$

where

$\Delta\sigma$. stress difference of a longitudinal reinforcement between two ends of a member in the yield mechanism assuring design, which equals $2\sigma_{yu}$ for a member with the planned yield hinges at both ends, equals $(\sigma_{yu}+\sigma_y)$ for a member with a planned yield hinge at the one end, and equals $2\sigma_y$ for a member without the yield hinges at both ends,

d_b diameter of the longitudinal reinforcement;

$\Sigma\psi$: total perimeter of the longitudinal reinforcement;

L: clear span;

b. width of the member section,

d: effective depth of the member section;

p_{wt}: required shear reinforcement ratio at the middle part of member;

σ_{wy}. yield strength of shear reinforcement at the middle part of member; and

ϕ : angle of the compressive strut of the truss mechanism at the middle part of member.

6.5 2 Bond Strength

Bond strength along the longitudinal reinforcement of columns and beams shall be calculated by Eq.(6.23) The bond strength for the top reinforcement of a beam shall be reduced to 0 8 times the value given by Eq (6.23)·

$$\tau_{bu} = (1.2+\frac{5p_w{'}b}{d_b})\sqrt{\sigma_B} \tag{6 23}$$

where $p_w{'}$: shear reinforcement ratio of the peripheral shear reinforcement

[Commentary]

Much amount of flexural reinforcement in beams and columns might cause bond splitting failure along flexural re-bars and lead to brittle failure, that should be prevented To assure the bond action for flexure mechanism [Refs 6 18, 6 40-6 42] and for truss mechanism [Ref. 6 43] are proposed as the methods to prevent bond failure. Design method in this design guideline is the combination of above two methods, that is, design bond stress according to flexure mechanism or truss mechanism should not be greater than bond strength Bond design is conducted for longitudinal flexural re-bars arranged at extreme edge layers

Design for bond provided in this section is simplified for practical design use based on conservative assumption Accurate design for bond might be conducted as described below.

(1) Design for a member without yield hinges or with a yield hinge at one end

In order to prevent bond splitting failure, one of bond stresses due to flexure, τ_f, or due to truss action, τ_t, should be less than bond strength, τ_{bu}

Bond stress due to flexure, τ_f, is defined as that produced by the steel stress difference between both ends of a member. These steel stresses would be obtained from flexural analysis at each end section. The τ_f is given by Eq. C6 26 considering inclined crack or yield hinge in tension side of a member. $\Delta\sigma$ is a stress difference between both ends of a member On a safety assumption, stresses of both steels in compression and tension of a member without intended yield hinges take their reliable strength, σ_y. Then $\Delta\sigma$ takes $2\sigma_y$ As for a member with an intended yield hinge at one end, steel stress in tension takes its upper bound strength, σ_{yu}, and steel stress in compression takes its reliable strength, σ_y, and $\Delta\sigma$ takes $\sigma_{yu}+\sigma_y$ It is not sure, however, that steel stress in compression always reaches σ_y, so it is allowed to get and use a working compressive stress of steel, σ_c, instead of σ_y, that would be obtained by flexural analysis using plain remaining plain assumption

$$\tau_f=\frac{0\ 25\pi d_b^2\Delta\sigma}{\pi d_b(L-d)}=\frac{\Delta\sigma d_b}{4(L-d)} \qquad \text{(Eq. C6.26)}$$

Alternatively, if bond stress required for truss mechanism defined in this design guideline is less than bond strength, equilibrium of truss mechanism is satisfied and then bond splitting failure might be prevented The bond stress required for truss mechanism is obtained from the equilibrium of forces illustrated in Fig C6 1 as Eq C6 27

$$\tau_t=\frac{V_t}{j_t\Sigma\psi}=\frac{bj_tp_{wt}\sigma_{wy}\cot\phi}{j_t\Sigma\psi}=\frac{bp_{wt}\sigma_{wy}\cot\phi}{\Sigma\psi} \qquad \text{(Eq C6.27)}$$

where

V_t · shear carried by the truss mechanism;

p_{wt} : shear reinforcement ratio at the central part of a member, which is not the actual ratio for the part but the amount required to resist the design shear; and

$\Sigma\Psi$: total perimeter of the tensile steel bars

The bond strength, τ_{bu}, would be obtained as following procedure [Ref 6 44]. The bond stress for upper steel in a beam should be reduced by 0 8

$$\tau_{bu}=\tau_{co}+\tau_{st} \qquad \text{(Eq C6.28)}$$

In Eq C6 28, τ_{co} presents a contribution of concrete and is given as:

$$\tau_{co}=(0.4b_i+0.5)\sqrt{\sigma_B} \qquad \text{(Eq. C6.29)}$$

where b_i represents the coefficient related to the mode of bond failure given by the smaller between b_{ci} and b_{si}.

Values of b_{ci} and b_{si} are given as

$$b_{ci}=\frac{(2\sqrt{2}d_c-d_b)}{d_b} \qquad \text{(Eq. C6.30)}$$

$$b_{si}=(b-\Sigma d_b)/\Sigma d_b \qquad \text{(Eq. C6.31)}$$

where

b_{ci} : coefficient for the corner splitting mode [see.Fig. C6.22];

b_{si} . coefficient for the side splitting mode [see.Fig C6.22] including the face-side splitting mode;

d_c · depth of the cover concrete from the center of the corner steel;

d_b : diameter of the corner steel;

Σd_b : total of diameter of the steel in a layer; and

b : section width of the member.

In Eq. C6.28, τ_{st} presents a contribution of shear reinforcement, and each value for corner splitting mode and for side splitting mode is given by Eqs.C6.32 to C6.34, respectively.

In case of the corner splitting ($b_i=b_{ci}<b_{si}$):

$$\tau_{st}=\frac{50A_w\sqrt{\sigma_B}}{sd_b} \qquad \text{(Eq. C6.32)}$$

In case of the side splitting ($b_i=b_{si}<b_{ci}$) with condition $N_t/2\geq N_u$:

$$\tau_{st}=\frac{(20/N_t+5N_u/N_t+15N_s/N_t)p_w'b\sqrt{\sigma_B}}{d_b} \qquad \text{(Eq. C6.33)}$$

In case of the side splitting ($b_i=b_{si}<b_{ci}$) with condition $N_t/2<N_u$:

$$\tau_{st}=\frac{(5p_w'b\sqrt{\sigma_B})}{d_b} \qquad \text{(Eq. C6.34)}$$

where

s : spacing of the shear reinforcement other than that in the hinge region,

A_w : sectional area of the shear reinforcement covering the corner steels (peripheral hoop or stirrup),

N_s : number of the flexural steels directly hooked by supplemental ties;

N_u : number of the flexural steels not hooked as above,

N_t · total number of the flexural steels ($=2+N_s+N_u$), and

p'_w · shear reinforcement ratio other than that in the hinge region, only shear reinforcement arranged at the extreme external side (peripheral shear reinforcement = $2A_w/(b\ s)$) can be counted.

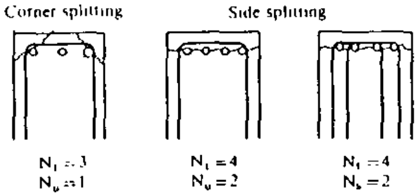


Fig. C6.22 Corner splitting and side splitting modes including the face-side splitting.

Equation C6 33 is derived from Eqs. C6.35 and C6.36 in Ref 6.44

$$\tau_{st,1}=1\ 22\frac{15.9A_w\sqrt{\sigma_B}}{sd_b} \qquad \text{(Eq C6 35)}$$

$$\tau_{st,2}=1.22\frac{7.63A_w\sqrt{\sigma_B}}{sd_b} \qquad \text{(Eq C6 36)}$$

where

$\tau_{st,1}$: τ_{st} for the steel that are directly hooked by shear reinforcement,

$\tau_{st,2}$ · τ_{st} for the steel that are not hooked by shear reinforcement; and

1.22 : coefficient used in the cases for steel bars other than the upper steel in a beam.

Test specimens in Ref. 6.44 have four flexural steel in a line, and two of them are hooked by peripheral lateral reinforcement ($N_t=4$, $N_u=2$). A_w in Eqs. C6.35 and C6.36 are replaced by p'_w, and then Eqs C6 37 and C6 38 are derived.

$$\tau_{st,1}=1.22\frac{15.9p_w'b\sqrt{\sigma_B}}{2d_b}\cong\frac{10p_w'b\sqrt{\sigma_B}}{d_b} \qquad \text{(Eq. C6 37)}$$

$$\tau_{st,2}=1.22\frac{7.63p_w'b\sqrt{\sigma_B}}{2d_b}\cong\frac{5p_w'b\sqrt{\sigma_B}}{d_b} \qquad \text{(Eq C6 38)}$$

When supplemental tie is used with the same sectional area of A_w, τ_{st} for intermediate bar hooked by supplemental tie is given as the summation of Eqs. C6 37 and C6.38.

$$\tau_{st,3}\cong\frac{15p_w'b\sqrt{\sigma_B}}{d_b} \qquad \text{(Eq. C6.39)}$$

Equation C6.33 takes the average of these three values, Eqs C6.37 through C6 39.

In checking upper steels in a beam, steels of a floor slab in orthogonal direction to the axis of the beam could be countable as a shear reinforcement not hooking them. Then τ_{st}' by Eqs. C6.40 or C6.41 could be added to Eqs C6 32 or C6.33, respectively.

In case of the corner splitting·

$$\tau_{st}'=20\frac{(A_{st}/s_{st}+A_{sb}/s_{sb})\sqrt{\sigma_B}}{d_b} \qquad \text{(Eq. C6 40)}$$

In case of the side splitting:

$$\tau_{st}'=20\frac{(A_{st}/s_{st}+A_{sb}/s_{sb})\sqrt{\sigma_B}}{\Sigma d_b} \qquad \text{(Eq. C6.41)}$$

where

A_{st}, s_{st} : area and spacing of the top steel in a slab; and

A_{sb}, s_{sb} . area and spacing of the bottom steel in a slab

The bond stress, τ_{bu}, obtained from Eq 6.23 is the worst case of Eq. C6 34 where many flexural steels are arranged in a layer and only two of them are hooked at the corner by shear reinforcement. Equation 6 23 will be derived as following. Depth of cover concrete measured from the surface of a steel and clear distance between flexural steels are specified not to be less than 1.5 times and 1.7 times their diameters, respectively Equation C6.42 is from this minimum requirements for steel arrangement. According to above specifications, minimum required section width is obtained as

$$b\geq2(1.5d_b)+(N_t-1)1.7d_b+N_td_b=2.7N_td_b+1.3d_b \qquad \text{(Eq C6.42)}$$

Equation.C6.42 could be simplified as $b/N_td_b=b/\Sigma d_b>2.5$, and then $b_i=b_{si}\geq1.7$ is obtained from Eq. C6 31 The contribution of concrete for this worst case is obtained from Eq. C6.29 as $\tau_{co}\geq1.18\sqrt{\sigma_B}\cong1.2\sqrt{\sigma_B}$.

Equation 6.23 is given as the summation of this minimum assurance value of τ_{co} and Eq C6 34. This design method for bond has already been applied at the time of verification for shear strength equation (see.Fig C6 5) , that is, in Fig. C6.5 the specimens that may fail in bond splitting have been excluded

(2) Design for a member with the yield hinges at both ends

As for a member with intended yield hinge at both member ends, two design conditions are provided, and it is necessary for the design for bond to satisfy one of them One, [a], is the condition to limit bond slip within small value (less than the bond slip at peak bond stress on bond stress versus slip curve), and the other, [b], is the condition to remain either flexural mechanism or truss mechanism in large bond slip situation (after peak on bond stress versus slip curve) [Ref. 6 42]

[a] Bond stress due to flexural action given by Eq C6 43, τ_f, should not more than the bond strength, τ_{bu}, given by Eq C6.28.

$$\tau_f=\frac{d_b\Delta\sigma}{4(L-d)} \qquad \text{(Eq C6.43)}$$

where $\Delta\sigma=2\sigma_{yu}$

[b] Smaller bond stress which is given by Eq. C6 44 or Eq C6 45 should not be more than the ultimate bond stress [Refs 6.42,6 45], τ_{bus}, after considerable bond slip. Generally, bond stress slip curve has a almost constant stress region (plateau) in large slip situation after peak if certain amount of lateral reinforcement is arranged The value of τ_{bus} gives the bond stress in such region. This alternative method is reflecting that a member could resist shear by existing of bond stress necessary for truss mechanism or flexure mechanism even in such region.

$$\tau_f=\frac{V_t}{j_t\Sigma\varphi}=\frac{b\,p_{wt}\sigma_{wy}\cot\phi}{\Sigma\varphi} \qquad \text{(Eq C6.44)}$$

$$\tau_f=\frac{d_b\Delta\sigma'}{4(L-d)} \qquad \text{(Eq C6 45)}$$

Ultimate bond stress, τ_{bus}, is given as follows

In case of the corner splitting ($b_i=b_{ci}<b_{si}$)

$$\tau_{bus}=\left(\frac{70A_wb}{sd_b}+0.4\right)\sqrt{\sigma_B} \qquad \text{(Eq. C6.46)}$$

In case of the side splitting ($b_i=b_{si}<b_{ci}$) with codition $N_t/2\geq N_u$:

$$\tau_{bus}=\left(\frac{60p_w'b(N_s+2)}{N_t\Sigma d_b}+0\ 4\right)\sqrt{\sigma_B} \qquad \text{(Eq. C6.47)}$$

In case of the side splitting ($b_i=b_{si}<b_{ci}$) with codition $N_t/2<N_u$:

$$\tau_{bus}=0.4\sqrt{\sigma_B} \qquad \text{(Eq C6.48)}$$

In Eq C6.45 $\Delta\sigma'$ is the difference of stresses of steels between both ends in a member, and is given by Eq. C6 49

$$\Delta\sigma'=\sigma_{yu}+\sigma_c \qquad \text{(Eq C6.49)}$$

where σ_c, compressive stress of steel, is obtained assuming plain remaining plain assumption at the overstrength limit design and using upper bound tensile strength of steel σ_{yu}.

Equation C6.47 is derived as following. The ultimate bond stress in Ref. 6.45, τ_{bus}, is given by Eq C6 50.

$$\tau_{bus}=1\ 22\left(\frac{23\ 3p_w'b}{\Sigma d_b}+0.3\right)\sqrt{\sigma_B} \qquad \text{(Eq. C6.50)}$$

Ultimate bond stress for flexural steel not hooked by shear reinforcement is given as Eq. C6.51, neglecting the effect of shear reinforcement in Eq. C6 50 ($p'_w=0$)

$$\tau_{bus}=1.22(0.3\sqrt{\sigma_B}) \qquad \text{(Eq. C6 51)}$$

Test specimens in Ref. 6.45 have four flexural reinforcement in a layer ($N_t=4$, $N_u=2$), then τ_{bsu} is estimated considering the effect of shear reinforcement in Eq. C6.50 by two times (see Eqs.C6.35 and C6.36).

$$\tau_{bus}=1.22\left(2\,\frac{23.3p_w'b}{\Sigma d_b}+0.3\right)\sqrt{\sigma_B} \qquad \text{(Eq. C6.52)}$$

The ultimate bond stress of Eq. C6.47 takes the average of the summation of Eq. C6.51 and Eq. C6 52.

Figures C6.23 and C6 24 show the results of verification for this design method for bond. Figure C6.23 shows the correlation of the assurance rotation angles obtained by both test and design equation without check of bond. Many specimens could not reach expected (calculated) rotation. While, the same discussion is done in Fig. C6 24 with check of bond by the method described in this section. Figure C6.24 shows the evidence of the method of design for bond. Horizontal axis in Fig. C6.24 denotes the yield drift for members in which bond failure is expected, where yield drift was calculated according to Ref 6 49.

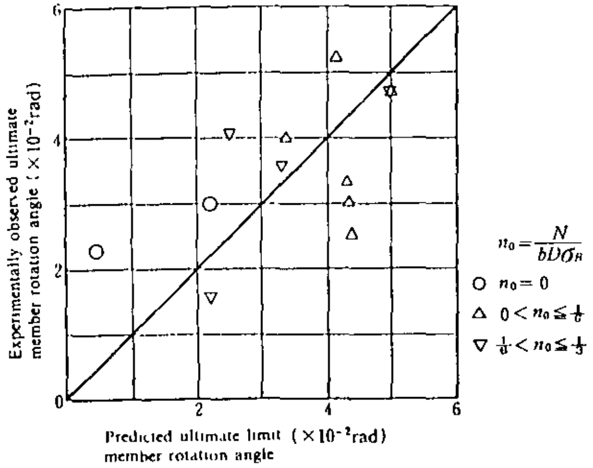


Fig. C6.23 Relation between the experimentally observed ultimate member rotation angles and predicted angles by use of Clauses 6.3.1 and 6.3.3 on the specimens that reveal failure in the bond splitting.

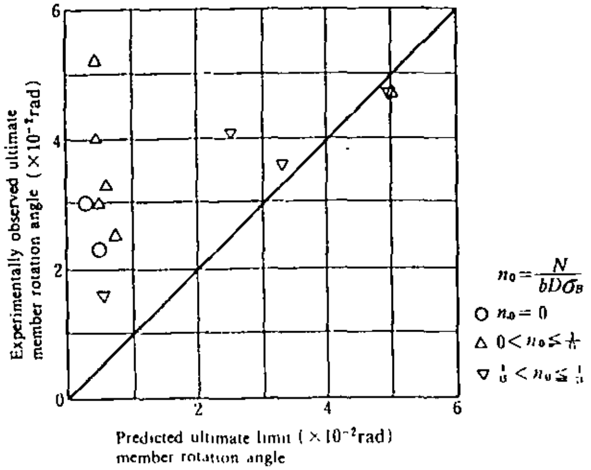


Fig. C6.24 Relation between the experimentally observed ultimate member rotation angles and predicted angles obtained from Eq. 6.21 , Eq. C6.32 and Eq. C6.33 on the specimens that failure in the bond splitting.

6.6 Design for Beam with Opening

The provisions in Clause 6 2 1 "Basic Principle," Clause 6 2 2 "Strength of Shear Reinforcement," and Clause 6.2 3 "Structural Requirements" shall apply to the shear design for a beam with an opening

[Commentary]

A beam with an opening reinforced transversely by only stirrups as shown in Fig. C6.25(a) is taken for a sample. Symbols used in the figure denote·

H : diameter of a hole or diameter of a circumcircle (if opening has a rectangular shape),

G · spacing of stirrups aside to hole; and

y_0 : distance from the center of a hole to the axis of a beam.

In the beam with opening arch mechanism is difficult to exist, and then shear shall be transferred by truss mechanism as shown in Figs C6.25(b) and (c) [Ref. 6 68]. The compression strut angle of concrete in upper and lower part of a hole is represented by ϕ_s in these figures. Symbols of horizontal arrows show bond stress, and vertical ones represent forces applied to concrete from stirrups. Unshaded portion shows the zone where diagonal compression stress field is not formed. Diagonal compressive stress of concrete around the hole becomes larger as the unshaded portion is extending. Effective depth for truss mechanism, j_{tw}, is defined as Eq. C6.53.

$$j_{tw}=j_t-\frac{H}{\cos\phi_s}-G\tan\phi_s \qquad \text{(Eq. C6.53)}$$

In Eq. C6.53, ($H/\cos\phi_s$ + G $\tan\phi_s$) represents the vertical height of the unshaded portion [see Fig. C6.25(c)]. Shear reinforcement aside to a hole should be provided within the range of $(j_{tw}/2+y_0)\cot\phi_s$ as shown in Fig. C6.25(a) Compressive stress of concrete in shaded portion is given by Eq. C6.54, where stirrup stress is assumed to be σ_{wy}.

$$\sigma_{cw}=p_{ws}\sigma_{wy}/\sin^2\phi_s=p_{ws}\,\sigma_{wy}\,(1+\cot^2\phi_s) \qquad \text{(Eq. C6.54)}$$

where p_{ws} denotes the shear reinforcement ratio aside to a hole.

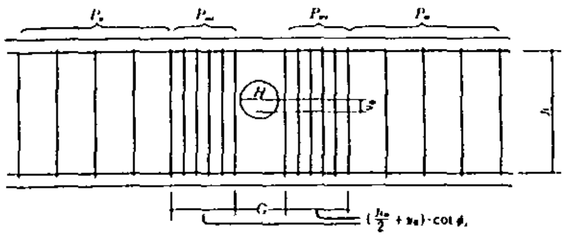
On the condition that σ_{cw} is equal to $\nu\sigma_B$ at the shear strength, ϕ_s is given by Eq C6.55.

$$\cot\phi_s=\sqrt{\frac{\nu\sigma_B}{p_{ws}\sigma_{wy}}-1} \qquad \text{(Eq C6 55)}$$

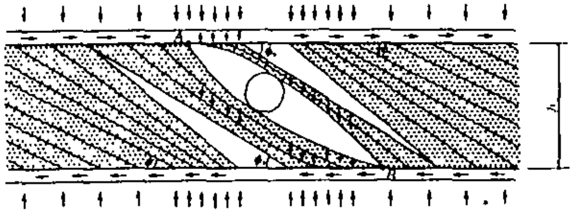
The value of $\cot\phi_s$ is allowed to take more than 2.0, because this case is limited in the vicinity of hole. Then reliable shear strength of a beam with opening is given by Eq C6 56.

$$V_u=b\,j_{tw}\,p_{ws}\,\sigma_{wy}\cot\phi s \qquad \text{(Eq C6 56)}$$

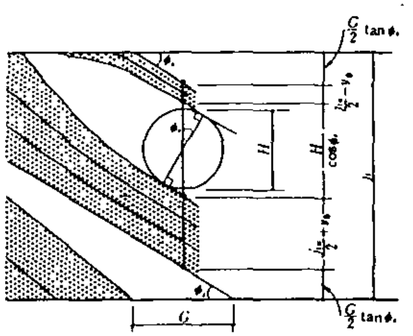
where $p_{ws}\sigma_{wy}$ = $(1/2-H/j_t)\nu\sqrt{\sigma_B})$ when $p_{ws}\sigma_{wy}>(1/2-H/j_t)\nu\sqrt{\sigma_B}$, which is the maximum amount of reinforcement for shear.



(a) Reinforcement arrange.



(b) Truss action



(c) Truss near a hole.

Fig. C6.25 Beam reinforced only by stirrups.

The attainable maximum shear strength of beam (reinforced up to its limit) with several opening ratios , H/j_t, are shown in Fig. C6 26. Here G is assumed to be equal to I 2H For example when H/j_t is equal to 0.25 (j_t=0.8D and H/D=1/5), the upper limit of $Vu/(bj_t\nu\sqrt{\sigma_B})$ takes 0 24. It means the upper limit by reinforcement of stirrups. The verification for this strength equation against test data [Refs. 6.69–6.73] has done and the results are shown in Fig. C6.27. Good prediction is obtained by this strength equation.

Inclined reinforcement whose development are anchored outside of stirrups aside to a hole, where compressive stress of concrete is not so high, as shown in Fig C6.28, can work well in its tension side Then the effect of inclined reinforcement given by Eq C6 57 could be added to the strength, V_u, of Eq C6.56

$$V_x = A_x\sigma_{xy}\sin\theta_x \qquad \text{(Eq. C6.57)}$$

where

θ_x : angle of inclined reinforcement to axis of a member, and

A_x : tension area of inclined reinforcement.

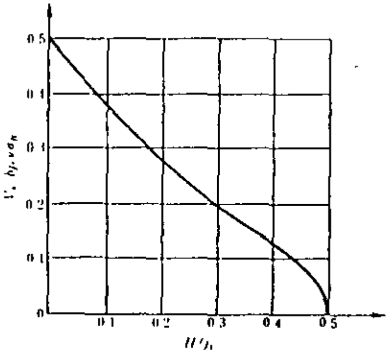


Fig. C6.26 Effect of the size of hole on the upper shear strength.

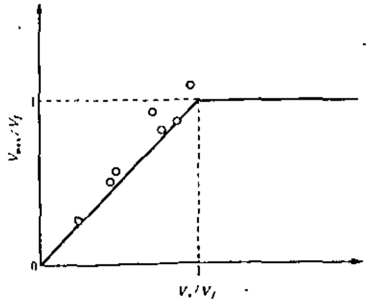
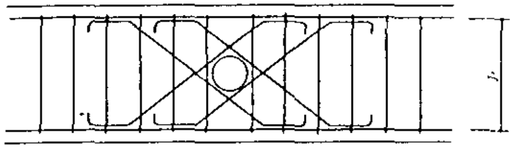
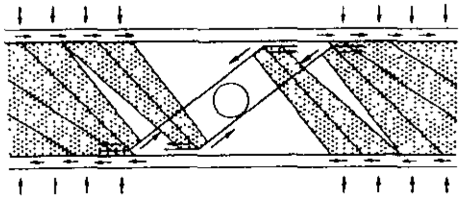


Fig. C6.27 Verification of strength equation.



(a) Reinforcement arrange



(b) Truss action

Fig. C6.28 Beam reinforced by the inclined reinforcement.

In a beam with top and bottom sub-beams reinforced longitudinally and laterally, each sub-beam have a truss mechanism as shown in Fig C6 29(b) Then shear strength could be obtained by Eq C6.58

$$V_u = 2bj_{ts}p_s\sigma_{sy}\cot\phi_s \qquad \text{(Eq C6 58)}$$

where $p_s\sigma_{sy} = \nu\sigma_B/2$ (when $p_s\sigma_{sy} > \nu\sigma_B/2$), which is the maximum amount of reinforcement for shear, and in which,

j_{ts} : distance between longitudinal reinforcements in the top and bottom sub-beams;
p_s : shear reinforcement ratio of the stirrups in the top and bottom sub-beams; and
σ_{sy} : yield strength of the stirrups in the top and bottom sub-beams

The value of $\cot\phi_s$ takes the smaller value in Eq. C6.60·

$$\cot\phi_s = 2$$

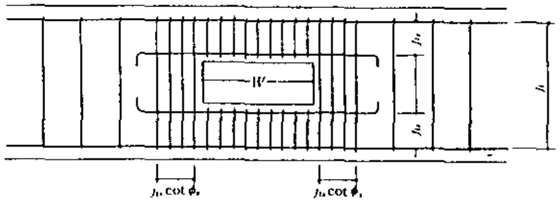
$$\cot\phi_s = \sqrt{\frac{\nu\sigma_B}{p_s\sigma_{sy}} - 1} \qquad \text{(Eq C6 60)}$$

Stirrups in top and bottom sub-beams should be provided in the range of $j_{ts}\cot\phi_s$ from the edge of the opening as shown in Fig C6.29(a). Inside longitudinal reinforcement in sub-beams should be anchored at the outside of extended stirrup arrangement from sub-beams with hooks bent inside of beam. Required strength of inside longitudinal reinforcement in sub-beams, $a_s s_y$, is obtained from Eq C6 61

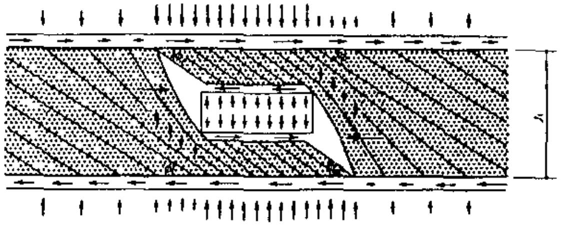
$$a_s\sigma_y = V_u W/(2j_{ts}) \qquad \text{(Eq. C6 61)}$$

where W denotes the length of opening And required strength of flexural reinforcement of extreme edge (outside longitudinal steels in sub-beams), $a_t\sigma_y$, should not be less than that by Eq C6 62.

$$a_t\sigma_y = \frac{V_u(W + j_{ts}\cot\phi_s)}{2j_{ts}} \qquad \text{(Eq C6 62)}$$



(a) Reinforcement arrange



(b) Truss action

Fig. C6.29 Beam with a rectangular opening reinforced by the longitudinal reinforcement and stirrups in sub-beams.

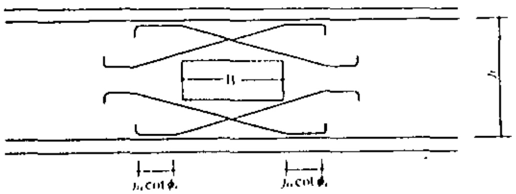
When the inclined reinforcement as shown in Fig. C6 30 are provided, the effect of its tension strength would be evaluated by Eq. C6 57, and would be added to V_u. Its anchorage should be done at the outside of additional stirrups arranged aside to opening The position of opening in a beam is desirable to be other than in yield hinge region. When a opening in yield hinge region should not be avoided, Eq 6 9 should be considered in a calculation of $\cot\phi$, and effectiveness factor, ν, should be given by Eq 6 8

Shear strength of a portion without hole should be obtained by Eq. C6.63, considering only truss mechanism

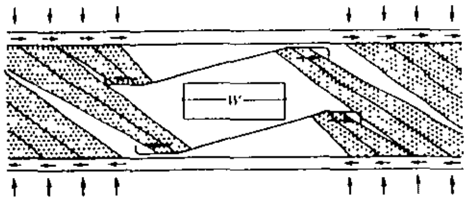
$$V_u = b j_t p_w \sigma_{wy} \cot\phi \qquad \text{(Eq. C6.63)}$$

where p_w denotes the shear reinforcement ratio in a portion without a hole, and the value of $\cot\phi$ should be calculated from Eqs 6.5-6.7 and 6 9

When a special steel would be used as a reinforcement for a hole, its effect, that should be verified by experiment and etc., might be added to shear strength Reference 6.74 is a research on this topic. In this case also, shear strength of a part without hole should be calculated by Eq. C6 63.



(a) Reinforcement arrange



(b) Truss action

Fig. C6.30 Beam with a rectangular opening reinforced by the inclined reinforcement.

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CHAPTER 7 : BEAM-COLUMN JOINT

7.1 Design Objectives

Beam-column joint shall be designed to maintain its integrity to the assuring deformation after the structure forms a yield mechanism The beam-column joint shall be designed to prevent a significant stiffness deterioration and slip-type hysteretic behavior by load reversals

[Commentary]

(1) Structural requirements in beam-column connection

It should be avoided in principle that a structure designed to form the overall yield mechanism fails in beam-column joints Because the beam-column joints would suffer from so large shear of adjacent beams and columns that it is difficult not only to design the joints for high ductility, but also to repair their damage There are many proposals regarding the resisting mechanism of a beam-column joint against shear. This chapter adopts a diagonal compression strut of concrete as the shear resisting mechanism, and the upper limit of design nominal shear stress in joint is provided to prevent failures.

The beam-column joint has to resist bending moment, shear, axial force and torsional moment produced by columns and beams at developing a yield mechanism without collapse, and also it is desired that the joint is without evident stiffness degradation Failures of a beam-column joint due to an anchorage failure of flexural reinforcement of beams or columns and due to compression failure of diagonal strut of concrete should be prevented However stiffness degradation of the joint and slip behavior in a story hysteretic characteristic may be resulted from shear cracks in the joint, yield of joint lateral reinforcements and bond deterioration in yield regions of column and beam bars. Because it is difficult to avoid the shear cracking and the bond deterioration of flexural reinforcements in a beam-column joint, the load vs deformation characteristics with slippage may be accepted in the guidelines so far as they do not affect so much on an earthquake response.

A recent study [Ref 7.1] indicated that the slipping hysteretic characteristics, as far as they were not so excessive, did not increase structural response in displacement range considered in the guidelines; i.e., an story drift angle of 1/100 radian. In this chapter, minimum shear reinforcement in a beam-column joint in proportion to joint shear stress is specified for preventing the excessive slipping hysteretic characteristics, and the design for embedment of flexural reinforcement of beams and columns and for anchorage of hooked reinforcement of beams are also described.

Beam-column joints in a structure designed under the concept of the guidelines are surely subjected to stresses corresponding to the flexural strength at yield hinges because the members adjacent to the joints are intended to form yield hinges The joints are the key elements for transferring forces between beams and columns and assuring a overall yield mechanism, therefore the design for beam-column joints is very important. However, the design criteria for joints specified in this chapter is not so severe than that for beams, columns and shear walls because severe design requirements would disconnect the continuity of the design for joints between the previous design method and the guidelines (this continuity is important in Japan) and make the scope of this design guideline narrow.

(2) Basic plan of beam-column joint

a) A beam-column joint is a segment where stresses of columns and beams are transferred by their longitudinal reinforcements through the core concrete of joint, therefore the joint is required to have enough strength and stiffness The core concrete in the joint must be confined by longitudinal reinforcements of columns and lateral reinforcements of the joint, and it is reasonable and practical to embed the longitudinal reinforcements of beams in the core In order to arrange the longitudinal reinforcements of beams inside the concrete core, beam widths are necessary to be less than the column width This treatment leads to avoiding large eccentricity between beams and columns. For example when beam widths are a half of a column width and the side faces of beams are not located outside the column side faces in the joint region, the maximum eccentricity between the beams and columns is less than 1/4 of the column width

b) When beams and columns are connected with eccentricity, torsional moments acting on the beam-column joint and on the column in the opposite direction to each other make torsional cracks in the joint and column, and reduce their stiffness A large eccentricity reduces a stress transferred to a side portion in the joint apart from the beam, then shear capacity of the joint decreases due to the reduction of effective volume resisting against shear.

There is no limit about the eccentricity in the design guidelines However, shear design for a beam-column joint with a large eccentricity may be done ignoring a portion outside the effective joint area and torsional design for the column due to the eccentricity is necessary.

c) A deep beam-column joint has a slender shape and a steep diagonal concrete strut. An elastic analysis of this kind of joint using a parameter of "a" (=beam depth/column depth) shows that shear stress distributions along the axis of column was expressed by a parabolic curve with its peak value at the center for "a"=1.0, by a rectangular with almost uniform value for "a"=1 5 to 2.0, and by a biquadratic curve with two peaks at the upper and lower parts for "a">2.0 [Ref.7.2] According to a few tests about a deep beam-column joint with "a"=2.0, shear failure concentrates to the upper part of the joint and the nominal shear strength is 60 % to 70 % of the joint strength with "a"=1.33 [Ref.7.3]. The depth ratio of beam to column has no limit in this guidelines, but a deep beam-column joint should be designed with caution [Ref. 7 4]

d) Chapter 10 requires structural slits (gaps) which separate non-structural walls such as spandrel walls from yield hinge regions of structural members. The stiffness of beams or columns increases due to those non-structural walls, consequently rotation of the yield hinges becomes so large that yield regions of their longitudinal bars will extend toward inside of the beam-column joint and bond deterioration of the bars may take place in the joint. However, the quantitative evaluation of the bond deterioration is not clear because there is few studies on this matter [for example, Ref. 7.5]

7.2 Design for Shear

7.2.1 Deign Principles

In shear design of a beam-column joint, reliable shear strength V_{ju} shall be greater than design shear V_j of the yield mechanism assuring design.

7.2.2 Shear Strength of Beam-Column Joint

Shear strength V_{ju} of beam-column joint shall be given by Eq. (7.1)

$$V_{ju} = \kappa \sigma_B b_j D_j \qquad (7.1)$$

where κ : factor dependent on shape of beam-column joint,equal to 0.30 for a cross-shape interior beam-column joint, and 0.18 for an inverted T-shape exterior beam-column joint and an L-shape top story exterior beam-column joint,

σ_B : compressive strength of concrete,

D_j : column depth or horizontally projected length of 90 degree hook; and

b_j : effective width of beam-column joint given by Eq. (7.2).

$$b_j = b_b + b_{a1} + b_{a2} \qquad (7.2)$$

where b_b, b_{a1}, b_{a2} denote the beam width, the smaller of one-quarter of column depth and one-half of distance between beam and column faces on the one side and another side of a beam.

7.2.3 Lateral Reinforcement

Lateral reinforcement ratio p_{jh} of beam-column connection shall be not less than 0.002, and shall satisfy Eq. (7.3)

$$p_{jh} \geq 0.003 (V_j / V_{ju}) \qquad (7.3)$$

[Commentary]

7.2.1 Design principle

(1) Beam-column joints should be designed against the severest stress combination of bending moments and shear forces at the yield mechanism assuring design. The location of yield hinges in a structure have been fixed at the design stage for the beam-column joints, then the non-hinge members including the joints should be designed for the stresses which are calculated by using the upper bound strengths at yield hinges. Hinge location of a beam is in principle at a face of column, but sometimes it is moved towards a center part of the beam in order to improve anchorage and bond of beam bars in the joint. In any case the upper bound strengths of yield hinges should be used for the design of joints.

The upper bound strengths of yield hinges are usually different in the loading direction; i.e., direction from right to left or left to right, then the joint should be designed against the severest condition.

The upper bound strength of yield hinge is defined as the upper bound flexural strength in the section 5.4. The section describes that the effects of 1) actual strength of longitudinal bars increasing from their nominal strength, and 2) reinforcement in floor slabs whose effective widths expand according to drift level of the story should be taken into account in calculating the upper bound flexural strength.

(2) Design shear force, V_j, for the beam-column joint in a structure with a strong column-weak beam mechanism is obtained by Eq. C7.1 [see Fig. C7.1]

$$V_j = T + C_s' + C_c' - V_c$$

$$= T + T' - V_c \qquad \text{(Eq. C7.1)}$$

In a T-shaped exterior beam-column joint, $C_s'+C_c'=T'=0$ is assumed. Shear from columns, V_c, is defined as the average of shear forces in the upper and lower columns, and is obtained by Eq. C7.2 using the upper bound strengths of beam hinges

$$V_c = 2 (l_b M_b/L + l_b' M_b'/L') / (l_c + l_c') \qquad \text{(Eq. C7.2)}$$

where M_b, M_b' : moments of left and right beams at the faces of column;

l_b, l_b' : span lengths of left and right beams;

L , L' : clear span lengths of left and right beams; and

l_c, l_c' : heights of upper and lower columns

The design shear force for the beam-column joint, V_j, at the top story calculated on the assumption that $V_c = 0$ gives safe side result.

As for a beam-column joint in a structure with yield hinges in columns, the upper bound strengths of column hinges dominate the design of the joint, so the design shear force, V_j, is given by exchanging beam forces for column forces in Fig. C7.1. The beam-column joint at the first floor level in a building with a basement floor should be designed using the upper bound strength of bottom end of the first story column and the design stress of top end of the basement column provided as a non-hinged member. For T-shaped joints in the top story or the bottom story, the design shear force of joint would be given by exchanging beam forces for column forces in inverted T-shaped exterior beam-column joints shown in Fig. C7.1. Axial force of column could be ignored because the shear force of beam-column joint is determined from bending moments and shear forces of adjacent members. The design forces produced by intermediate longitudinal reinforcement in a column with a yield hinge should be added to the forces of T and C' in Eq. C7.1 according to the intended rotation angle at the hinge region. The shear force calculated by above method is the vertical shear force, V_{jv}, and the lateral shear force, V_{jh}, could be obtained by Eq. C7.3 approximately, if necessary.

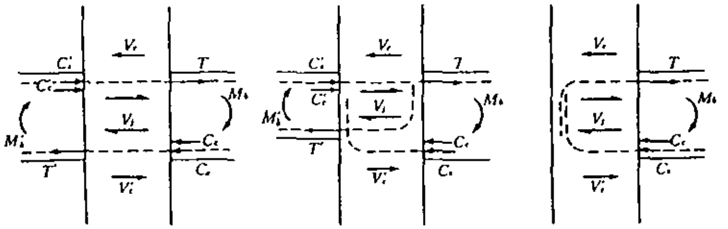


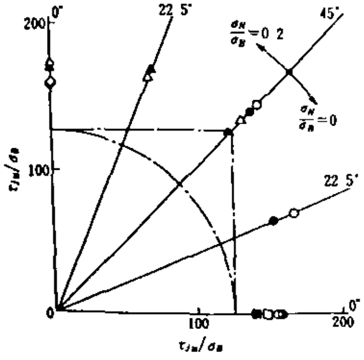
Fig. C 7.1 Internal stress resultant of joint.

$$V_{jh} = V_{jv} (D_b / D) \qquad \text{(Eq C7 3)}$$

where V_{jv} : vertical shear force;
D_b : depth of beam; and
D : depth of column.

(3) All beam-column joints at and above the ground floor level of a structure should be designed according to the provisions of this chapter. Similarly to the design for columns, the design forces are desirable to be determined considering lateral loads acting simultaneously in two directions. The shear force in a beam-column joint subjected to the lateral loads in two directions is estimated by superposing the forces at yield mechanism in each direction, and is larger than that obtained from one directional load

On the contrary, there are few research on the shear strength of beam-column joint subjected to any directional lateral loads Especially there are very few research on the behavior of shear failure in beam-column joint prior to yielding of the beams or columns As shown in Fig. C7.2, test results on three-dimensional interior beam-column joints subjected to bidirectional load reversals show that the shear strength of joint in 45 degree direction was larger than that in the main direction (beam axial direction) under no column axial force, but was not larger than that under large column axial force (σ_n / σ_B=0.2) [Ref. 7.6]. The bidirectional shear strength diagram of column is represented as an ellipse. If this characteristic could be applied to beam-column joints, it would not be necessary for design of joints to consider the strength reduction due to the effect of bidirectional loads. The shear strengths and the story drift capacities of three-dimensional beam-column subassemblages after yielding at the four beam ends do not show large differences between the beam axial direction and the 45 degree direction. Therefore the guidelines allow to design a joint only in each beam axial direction, when enough shear strength is provided to the joint. Further researches on the shear strength of joint in any direction are necessary



Loading type	axial stress ratio σ_N/σ_B	
	.0	0 5
Uni-direction	□ ■	◇ ◆
Bi-direction	○ ●	△ ▲

Open mark : forward loading
Solid mark : backward loading

—·— square lines and a circular arc interpolated between shear strength in two principal directions (0-degree) calculated by Eq 7 1

Fig. C 7.2 Bidirectional shear strength interaction of interior joint.

Since the joint surrounded by structural walls does not subject to so large shear in the direction of wall plane, it is not necessary to design for such joint The joint adjacent to a structural wall, such as boundary beam-column joint in a wall, would be designed as a part of a boundary column in a wall for lateral loads in direction of the wall plane, and be designed for the embedment of longitudinal reinforcement of the boundary beam, so that it is allowed to design such joints for the transverse lateral load independently

7 2.2 Shear Strength of Beam-Column Joint

Shear strength of beam-column joint should be based on the strength multiplying the upper bound shear stress dependent on the shape of beam-column subassemblage by the effective shear resisting area.

(1) Shear resisting mechanism

The shear resisting mechanism in beam-column joint can be considered to consist of a diagonal concrete strut action and a truss action [Ref 7.7]. This is almost the same shear resisting mechanism as for beams and columns. The diagonal concrete strut action is the stress transferring mechanism by a diagonal compression strut of concrete connecting a pair of compression zones at the corners of joint produced by bending moments of beams and columns. The truss action is the stress transferring mechanism by tensile resistance of lateral and vertical reinforcements in the joint and by compressive resistance of uniform diagonal concrete strut in the joint panel The truss action depends also on the bond stress transfer along the column and beam longitudinal reinforcements in the joint, so that bond deterioration reduces the stress transferred by the truss action Previous test data show that the effect of lateral reinforcement in beam-column joints on the shear strength are not so much and the failure of diagonal compression strut of concrete dominates the strength. Therefore the guidelines have the limitation of working shear stress of concrete to prevent shear failure.

Interior joint test results of 68 specimens, tested during the period of 1966 to 1988 in Japan and other countries, were compared with actual concrete strength in Fig. C7.3. Twenty-four specimens showed shear failure in joint prior to the yielding of beams (solid triangle symbols in Fig. C7.3; J-type), and forty-four specimens showed shear failure in joint after the yielding of beams (open circle symbols; BJ-type). The vertical axis is the nominal shear stress of joint, τ_{ju}, which is the amount of maximum strength divided by the product of b_j and D_j defined in the section 7.2.2. The value of b_j in J-type takes the average width of column and beam. The horizontal axis is compressive strength of concrete cylinder, σ_B. The range of σ_B for all specimens is from 100 to 550kgf/cm^{2}and that of J-type is from 100 to 400kgf/cm^2. The lower bound of τ_{ju} could be estimated to be 0.3 σ_B. Shear strengths of BJ-type are dominated by the flexural strength of beams. Though some of them were lower than 0.3 σ_B, shear failure occurred due to the increment of deformation. The deformations at shear failure in BJ-type specimens except for one specimen with very small shear span ratio were more than 1/50 in story drift which is the assurance deformation in the guidelines Therefore shear failure in joint could be avoided when the joint shear stress is kept to be less than the joint strength of J-type specimens.

Although the shear strengths of beam-column joints plotted in Fig C7 3 include the effect of shear reinforcement in them, the figure shows that the shear strengths increase mainly with compressive strength of concrete Equations described the relation of shear stress and specified concrete strength were proposed previously, but most of them underestimate the shear strength in the range of high compressive strength of concrete Equation 7 1 is proposed modifying this tendency, in which the shear strength of beam-column joint is proportional to the compressive strength of concrete considering that shear failure is due to compression failure of concrete strut The average ratio of τ_{ju} to σ_B is 0.345 and its deviation is 0 051 in J-type specimens The value of 0 3 is proposed as the coefficient "κ" in Eq. 7.1. Consequently more than 80% of interior beam-column joints designed by Eq 7.1 will have the shear strength of more than 0 3 σ_B

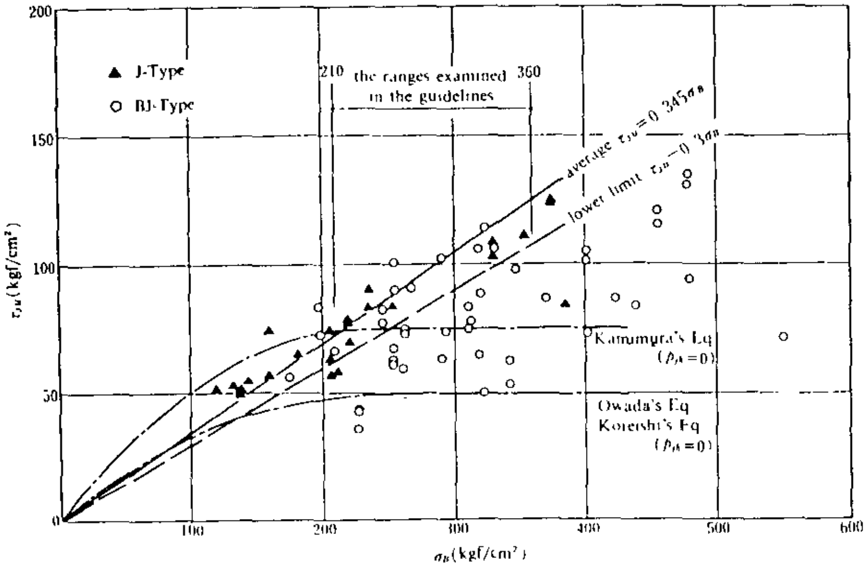


Fig. C 7.3 Average joint shear stress vs. compressive strength of concrete of interior joint with cross-shape subassemblage.

As for T-shaped exterior beam-column joints as well as interior joints, test data on shear strength are summarized in Fig C7 4 There are only five specimens of J-type and seven specimens of BJ-type Other specimens failed in anchorage of beam bars or in flexural yield of beams The average ratio of τ_{ju} to σ_B is 0.194 and its deviation is 0 013 Proposed value of κ in Eq. 7.1 is 0.18. More than 80% of exterior beam-column joints designed by Eq 7.1 will have the shear strength of greater than 0 18 σ_B.

The first order regression relationship among test data would be described as τ_{ju} = 0.249 σ_B+19.2 in kgf/cm^2 for the interior joints and as τ_{ju} = 0 105 σ_B + 21.9 in kgf/cm^2 for the exterior joints. Judging from these regression relationships, Eq 7 1 gives a high shear strength in the range of high compressive strength of concrete The reason why the relation of shear strength and concrete compressive strength is expressed with a proportional expression is that such type evaluation is simple and practical for structural design, then it should be noticed that the upper bound of the specified design compressive strength of concrete is 360kgf/cm^2 The coefficient of κ should not be used in the case that compressive strength of concrete is more than 360kgf/cm^2.

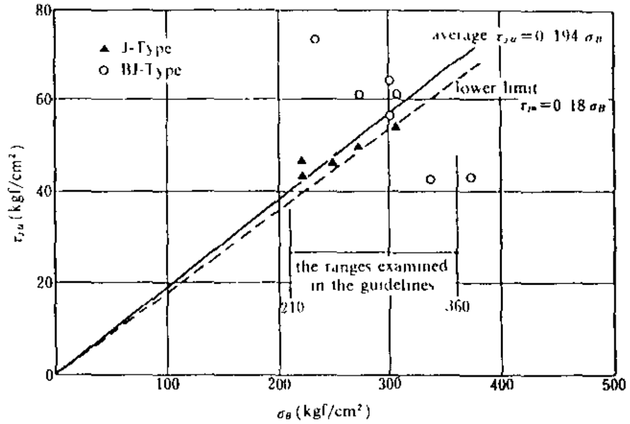


Fig. C 7.4 Average joint shear stress vs. compressive strength of concrete of exterior joint with a rotated T-shape subassemblage.

(2) Effective width b_j and effective depth D_j of beam-column joint

The effective width of joint, b_j, should be obtained by adding a beam width to an effective width of column [refer to Fig. C7 5]. The effective width of column is defined as the smaller of a half distance from beam side face to column face and one-fourth of the column depth on either side of the beam. In another words, the effective width, b_j, is based on the average width of beam and column, and is limited to the sum of beam width and one-fourth of column depth, assuming that joint stresses are effectively transferred through a zone covered by four lines with their angle of 1/2 radian from each side face of beam This concept considers that when the beam width is very narrow against the column width or the beam connects eccentrically to the column, some part of column volume in beam-column joint far from the side face of beam does not contribute to shear strength. According to this definition on the effective joint width, it takes the average width of the beam and the column, no matter how eccentrically the beam connects to the column, when the beam width is more than a half of column width for a square column And also it often takes the sum of the beam width and one-fourth of the column depth, when the beam width is less than a half of the column width or the beam connects to the longest side of the rectangular column. In the case that a width of beam or column is different from a width of the opposite beam or column of the joint respectively, or a couple of beams connect to a column with eccentricities different from each other, it is necessary to define another effective widths for those beam-column joints by understanding the formation of compression strut in the joints. For example, the joint with two different widths of beams takes the average width of them as b_b in Eq 7 2. Further researches on this field are required.

The effective depth for interior beam-column joint may take column depth, D. In previous researches on beam-column joints, the effective joint depth took conventionally the moment arm of column, j_c, ignoring the effect of axial force on the arm length It is seemed to be the same method as evaluating shear strength of beams or columns Shear strength of beam-column joint would depend on the diagonal compression strut of concrete which links both compression edges in the joint adjacent to the ends of beams and columns, so that it is reasonable to take the horizontal length of joint as the effective depth and also such manner is simple to calculate the strength The effective depth for a T-shaped exterior beam-column joint takes the horizontally projected length from the outside of 90-degree hook to the beam end or the horizontal length from the beam end to the end of steel anchor plate in case of using mechanical anchorage. This is based on the same concept taking the horizontal length of compression strut of concrete as the effective depth for the interior beam-column joint

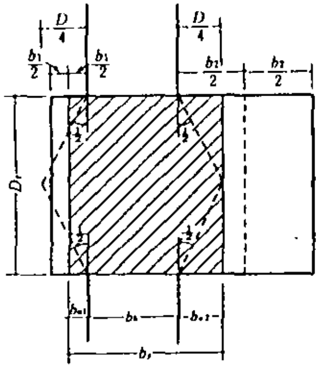


Fig. C 7.5 Defined effective width of beam-column joint, which is used in case of eccentric joint and/or different widths between beam and column.

(3) The confinement effect of transverse beams on beam-column joint

From the beam-column joint tests with non-loaded transverse beams it is well known that the transverse beams make the joint stiff and strong. Figure C7.6 shows the test data concerning the relationship between coverage ratio of a joint by transverse beams, λ, and shear strength magnification, β, of a joint with transverse beams to a joint without them [Ref. 7 8] The solid parabolic curve shows the relationship.

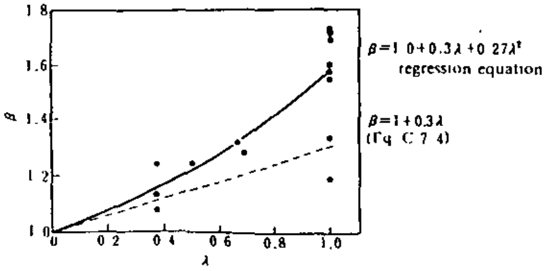


Fig. C 7.6 Relationship between shear strength magnification and coverage ratio by transverse beams.

In existing buildings, however, the beam-column joint with beam hinges subjected to bidirectional forces during an earthquake can not be confined so much by the transverse beams, because they also have yield hinges and cracks on the beam ends. The confinement effect by transverse beams on shear strength of a beam-column joint is not taken into account, because the guidelines treat a structure with a beam hinge mechanism. However, only if transverse beam bars are assured not to yield at the column face of the interior beam-column joint by forming a column yield mechanism or forming beam yield hinges located far from the column faces, the shear strength could be increased by the factor β expressed in Eq. C7 4

$$\beta = 1 + 0 3 \lambda \qquad \text{(Eq. C7.4)}$$

where $\lambda = \Sigma (b_{Lb} \cdot D_{Lb}) / 2 (D_b \quad D)$, in which b_{Lb}, D, D_b and D_{Lb} denote the width of transverse beam and depths of column, beam and transverse beam, respectively.

For the beam-column joint with a transverse beam in one direction, strength increment should not be allowed. As shown in Fig. C7 4, Eq. C7 4 gives the lowest prediction to scattering test data.

Equation 7.1 does not consider the effect of transverse beams on the shear strength of beam-column joint, because many beam-column joints in a structure designed by the guidelines have yield hinges at beam ends of all directions in principle, and the confinement effect of transverse beams cannot be expected. However interior beam-column joint test specimens with transverse beams after subjected to two times cyclic drift angle of 1/75 indicated a larger strength than that without transverse beams [Ref. 7.9]. And floor slabs also have the effect on confining the beam-column joint. So it is possible for transverse beams with yield hinges to increase a shear strength under low stress levels in the transverse beams The relation between stress level of transverse beams and application of Eq C7.4 should be discussed in future. The strength increment factor different from Eq. C7.4 may be used when the factor is obtained from an appropriate experiment.

7.2.3 Lateral Reinforcement

(1) Recent researches show that there is not so large effect of lateral reinforcement in a beam-column joint on its shear strength. Figure C7.7 plots the relationship between the strength ratio of shear strength to concrete strength, τ_{ju} / σ_B, and the product of the amount of lateral reinforcement and its strength, $p_{jh}\, \sigma_y$ The points linked by a solid line mean the specimens which were tested by the same researcher in a test series varied only the amount of $p_{jh}\, \sigma_y$. According to Fig. C7.7 it is very difficult to recognize the effect of lateral reinforcement on the shear strength except for a few cases. The guidelines make sure that the role of lateral reinforcement in a joint is to keep the stiffness and ductility of joint but not to increase the shear strength of joint. Then the minimum amount of lateral reinforcement is given in proportion to the ratio of design shear to shear strength of a beam-column joint.

Some foreign codes require more amount of lateral reinforcement than that required in this section, however it is difficult to arrange large amount of lateral reinforcement more than 0.3% by only hoop reinforcement and also is difficult to add sub-ties to beam-column joint against Japanese conventional construction method. This is the reason why the minimum amount of joint lateral reinforcement is 0 2%, and the amount of lateral reinforcement of 0.3% is required for the joint whose design shear stress is nearly equal to its shear strength. It is desirable to establish a reasonable design

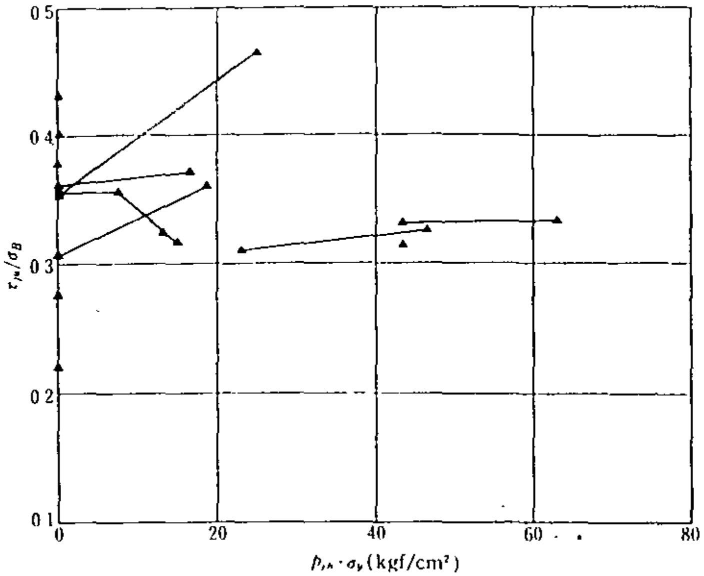


Fig. C 7.7 Shear strength of interior joint divided by concrete strength vs. product of lateral reinforcement ratio and its yield strength.

method for lateral reinforcement in a beam-column joint considering the effect of column axial force and bond strength, the required deformation capacity of yield hinge adjacent to the joint, and anchorage strength of hooked bars in T-shaped exterior joint

(2) Design for vertical reinforcement in a beam-column joint is not required in the guidelines, because usual columns would have at least a intermediate longitudinal bar in each column face and the bar works as the vertical shear reinforcement. The effect of vertical shear reinforcement on joint shear strength is not clear yet, but test results of T-shaped exterior joints show that the lateral and vertical shear reinforcement arranged with well balance and a large axial force of column prevent a shear failure of joints after yielding of the beams [Ref 7.12]. The intermediate column bars of joints with yield hinges in beams are useful to resist joint shear force because the bars are not subjected to so large tension by the column end moments, but the bars of joints with yield hinges in column may not be useful. Therefore it would be necessary to arrange additional lateral shear reinforcement instead of the vertical shear reinforcement for the joints of the latter

7 3 Anchorage of Beam and Column Reinforcements

7.3.1 Anchorage Method

Longitudinal reinforcement of beam with an intended yield hinge, as a general rule, shall pass through the column core of the beam-column joint or shall be anchored into column core of the beam-column joint with a 90 degree hook Development length of a bar shall be counted from the critical section at the column face or at the top and bottom beam faces

7 3.2 Anchorage with 90-Degree Hook

Development length shall conform to requirements in Chapter 9. Beam longitudinal reinforcement shall be extended beyond the mid-depth of column with a 90-degree hook Extension of a 90 degree hook shall be placed in the beam-column joint.

7.3.3 Bars Passing through Joint

When longitudinal reinforcement of beams or columns with intended yield hinges at both faces of joint passes through the joint, bar size to member depth ratio shall be determined not to cause significant stiffness reduction or slip-type hysteretic behavior under load reversals.

[Commentary]

7.3.1 Anchorage Method

(1) Longitudinal reinforcement of beam normally passes through an interior joint and is anchored with 90-degree hook in an exterior joint. It is desirable to avoid the bond deterioration, because yielding of the beam longitudinal reinforcement is liable to penetrate into the joint and to deteriorate bond resistance along the reinforcement, consequently it causes also deterioration of hysteretic energy dissipation. The bond deterioration may be delayed in the interior joint by passing the beam reinforcement in the confined concrete core, and by limiting the bond stress level in design. The limitation of bond stress level is specified in 7 3.2 and 7.3.3.

(2) Joint cover concrete adjacent to a loaded beam may spall out or fail in punching shear with cone-shape by tension of beam reinforcement, resulting in a reduction of development length The development length is necessary to be evaluated at the face of core concrete in joint if such phenomenon would occur from early stage. Anchorage capacity of a bar subjecting to compression force does not reduce so much than that under tensile force.

This guidelines adopts the way as the critical section for calculating the development length of a bar is taken at the face of column conventionally, because spalling of cover concrete or reduction of bond resistance after bar yielding is affected by many factors and these behaviors are not so clear.

7.3.2 Anchorage with 90-degree Hook

(1) According to T-shaped exterior beam-column joint tests where beam bars were anchored with 90-degree hooks, local compression failure of concrete inside the bar bends or splitting failure of cover concrete beside the bar bends were reported as the failure mode of anchorage, even if the difference of failure modes between anchorage and joint shear were not clearly distinguished The

failure of anchorage should be prevented, because it causes the rapid strength reduction and the poor energy absorption in hysteretic characteristics

The required development length is specified in Chapter 9. According to recent studies, strength of anchorage depends on the horizontal projection length of a beam bar including the bend portion, but it does not relate to the vertical extension of the beam bar beyond 12 times bar diameter, d_b Then it is better to locate a hook as outside as possible in order to get as long horizontal development length as possible and to extend the beginning of bent at least beyond the mid-depth of column.

The strength of anchorage with hook is affected by many kinds of factors, such as a radius of hook and a thickness of cover concrete in a beam-column joint. Some of the proposed equations on anchorage strength are based on the strength estimated by the sum of bond resistance along the horizontal development and anchorage resistance at the bent portion. Equation C7.5, however, evaluates a local compressive strength of concrete, f_{bear}, at the inside of hook as the anchorage strength, P, assuming that bond deterioration of horizontal part of a bar will occur soon after the yielding of beam [Ref. 7.13].

$$P = w\, d_b\, f_{bear} \sin\theta\ h / (h-j) \qquad \text{(Eq. C7.5)}$$

where $w = \beta\sqrt{2}\ r \cos(\pi / 4 - \theta)$

$\theta = \tan^{-1}(l_{dh} / j)$

$l_{dh} = l_1 + r + d_b$

$\beta = (r / 3d_b)^{-0.84}$

$f_{bear} = \alpha\, \gamma \sqrt{\sigma_B}$

$\alpha = 16.1\ C_0 / d_b$

$\gamma = 1 + 30\ A_s / (l_1 s)$,

in which d_b : diameter of bar,

h : distance of horizontal support of column,

r : radius of hook,

σ_B : compressive strength of concrete;

l_1 : development length of straight portion;

A_s : sectional area of lateral reinforcement;

s : spacing of lateral reinforcement;

j : moment arm of beam; and

C_0 : thickness of cover concrete from column face to center of bar. All parameter above are measured in kgf and cm

The average ratio of experimental strengths to calculated strengths by Eq. C7.5 is 0.95 and its deviation is 0.18. The test data used in this evaluation consist of those of 73 specimens Equation C7 5 is provided only for the bar at the extreme edge, and further discussion is necessary on the evaluation of group effect of multi-bars including intermediate bars.

(2) To prevent congestion of bar arrangement in a beam-column joint with bent down hooks for top bars and bent up hooks for bottom bars of a beam, U-shaped bar arrangement is used practically in which both the ends of the top and bottom bars are connected in the joint [Ref. 7.15]. Sometime it occurs that an extension of hook is placed below/above a beam-column joint or enough horizontal projection length cannot be provided within the joint. In these cases, shear failure or anchorage failure in the joints is prevented by ways that transverse reinforcement placed at just inside of the hook can resist against the large local compression due to the hook and the lateral reinforcement placed beside the hook can transfer the beam tension to the backside of column [Ref. 7.16].

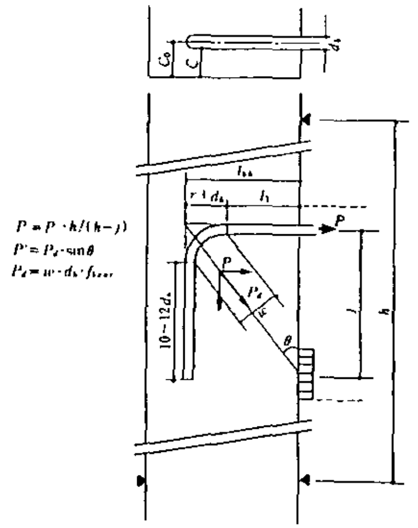


Fig. C 7.8 Definition of variables used in Eq. C 7.5 which evaluates anchorage strength of 90-degree hooked beam bar.

Cover concrete on backside of column spalls off so easily that the extension of hook reduces its anchorage strength in case that the extension is placed in the same layer as the extreme bars of column. There is no requirement about the thickness of cover concrete behind the extension of hook, because of few research on it Some experiments showed the punching shear failure at the part between the column bars, when the beam bars subjected to compression force. Another experiment in which specimens had adequate distance between the layers of extension and extreme column bar (the distance was more than the spacing of column bars) prevented the anchorage failure [Ref 7.17]. Even in this case, it is desirable that the column bars near by the extension of hook are confined by hoops or sub-ties. It had better keep the development length rather than keep the thickness of cover concrete behind the extension of hook, when it is difficult to keep enough horizontal projection length against the column depth.

7.3.3 Bars Passing Through A Beam-column Joint

The average bond stress in a beam-column joint, τ_a, of beam longitudinal reinforcement passing through the joint is described by Eq C7 6

$$\tau_a = a_s \, \Delta\sigma_s / D \, \phi_s \qquad \text{(Eq C7.6)}$$

where $\Delta\sigma_s$. stress difference of a bar at the both faces of a joint;

D depth of column;

a_s : cross sectional area of bar, and

ϕ_s · perimeter of bar

Using a diameter of the bar, d_b, instead of a_s or ϕ_s, Eq C7 6 is modified to Eq C7.7.

$$\tau_a = \Delta\sigma_s \, d_b / 4D \qquad \text{(Eq. C7.7)}$$

Beam longitudinal reinforcement in a beam-column joint with beam yield hinges at the ultimate strength stage reaches its tensile yield strength at the joint face, but does not reach its compression yield strength under the condition of large reinforcement ratio of tension bars to compression bars. In case both the tension and compression bars yield, the stress difference of a beam bar at both the faces of joint can be presented as $2\sigma_{yu}$, using the steel strength for calculating the upper bound bending strength, σ_{yu} Bond strength is assumed to be proportional to the square root of the compressive strength of concrete, then bond strength, τ_a, is described as $\gamma\ \sqrt{\sigma_B}$ And substituting these relations to Eq. C7 7, Eq. C7 8 is obtained as follow·

$$D/d_b = \sigma_{yu} / (2\gamma\ \sqrt{\sigma_B}) \qquad \text{(Eq C7 8)}$$

Here assuming that μ is equal to 2γ, Eq C7 9 is driven as the proposed design equation.

$$D/d_b \geq \sigma_{yu} / (\mu\ \sqrt{\sigma_B}) \qquad \text{(Eq C7 9)}$$

There is no recommendation for the value of μ in the design guidelines. The New Zealand design code recommends some values to prevent the pulling out of bars certainly from a joint with yield hinges at its both faces. It is very difficult to prevent beam bars from being pulled out because of the conventional combinations of material strength, bar size and column size used usually in Japan, and then some bond deterioration is allowed in the guidelines.

The bond deterioration and pulling out of beam longitudinal reinforcement from a joint cause slip-type hysteretic behavior with poor energy absorption capacity, and also they are accompanied with the following problems· (1) large response drift; (2) large pulling out deformation prior to flexural yielding, (3) early compression failure of concrete at beam end due to large rotation at the beam end; and (4) difficulty to repair the failure of pulling out.

However, since the bond deterioration in a joint expands gradually with increment of displacement, it is expected that large and sudden strength reduction will not occur even though the bond has been lost completely in the joint, if beam bars are assured to be anchored in some places in the both beams and the compression zones of beam ends are well confined to prevent the compression failure

The recent research [Ref. 7.1] led the μ value in Eq C7 9 with the following process· (1) It was ascertained by the response analysis with slip-hysteresis model that hysteretic energy absorption capacities do not affect so much on earthquake responses; (2) the index of allowable slip characteristic was defined as the equivalent damping coefficient, h_{eq}, of 0.1 or more at the drift angle of 1/50; (3) the relationship between h_{eq} and bond index, τ_s (= σ_{yu} d_b / 2D) was investigated using experimental results; and (4) the factor of μ was proposed to be 10 0

TABLE C 7.1 MINIMUM COLUMN DEPTH WITH BEAM BARS PASSING THROUGH A JOINT IN ORDER TO PREVENT ITS BOND DETERIORATION, SATISFYING A CONDITION OF $D/d_b \geq \sigma_{yu}/10\sqrt{\sigma_B}$

SD35	D22	D25	D29	D32	D35	D38	D/d_b
σ_B = 210	67	76	88	97	106	115	30 2
240	63	71	82	91	99	108	28 2
270	59	67	78	86	94	102	26 6
300	56	64	74	81	89	96	25 2
330	53	61	70	78	85	92	24 1
360	51	58	67	74	81	88	23.1

SD40	D22	D25	D29	D32	D35	D38	D/d_b
σ_B = 210	76	87	101	111	121	132	34 5
240	71	81	94	104	113	123	32.3
270	67	77	89	98	107	116	30 4
300	64	73	84	93	102	110	28 9
330	61	69	80	89	97	105	27.5
360	58	66	77	85	93	101	26 4

[Note]

Values of column depth are rounded up.

Upper bound strength of beam bars are as follows

for SD35 bars · $\sigma_{yu} = 1.25 \times 3500 = 4375$ kgf/cm^2; and

for SD40 bars : $\sigma_{yu} = 1\ 25 \times 4000 = 5000$ kgf/cm^2.

It is noticeable that the earthquake response drift is not so much affected by the hysteretic energy absorption capacity, but the number of reversals with large response drifts increases.

Applying 10.0 for μ in Eq C7.9, the required column depths, D, are listed in Table C7.1 in the relationship among d_b, σ_{yu}, and σ_B. In the table, the upper bound material strength is assumed for σ_{yu}, that is the stress of longitudinal beam bar at the yield mechanism assuring design

According to Table C7.1, the column depths have become unreality in case of using large size of beam bars and the combinations of column depth and beam bar size commonly used in the current design are strictly restrained. These are from using the upper bound strength of steel and for assuring the yield mechanism. However, it is allowed for practical use to ease this restriction listed in Table C7.1 by the following reasons· (1) Dynamic response of drift using the upper bound strength is smaller than that using the reliable strength, (2) bond deterioration does not lead to serious problems comparing with the strength reduction due to shear failure, and (3) some connections are allowed to ease this restriction if the restriction is satisfied on the average in a whole structure (or at least in every story). For the time being, μ is allowed to take 12.5 for practical use Then 0.8D in Table C7.1 is allowed to be used.

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CHAPTER 8 : FOUNDATION STRUCTURE

8.1 Design Actions

Design forces for foundation structures shall be determined from the design forces used for the yield mechanism assuring design of the super structure, and vertical and horizontal forces acting on both the ground floor and underground structures. Design of a foundation girder shall consider reactions that act on the bottom face of the girder according to its rigidity.

[Commentary]

Foundation structures shall safely transmit design vertical and lateral actions from the super structure to the underground structure. Since the foundation structure shall have higher structural performance than the super structure, and in principle it shall not show failure mechanism when the super structure develop a yield hinge mechanism, design loads for a foundation structure should include both the design vertical and lateral load added to the design forces used for the yield mechanism assuring design of the super structure.

As an exceptional case, a yield hinge of foundation structure shall be allowed as the total yield hinge mechanism. Load carrying capacity at the yield hinge mechanism should be clearly calculated in this case, and the foundation structure should be provided with sufficient ductile. For an example, in case that a shear wall is directly supported by a footing system, the ultimate capacity by uplifting of the shear wall is clearly estimated [Ref. 8.1], and then the total yield hinge mechanism associating with the uplifting of the shear wall can be allowed. The foundation girders or beams, however, should be designed in order to be provided with sufficient ductility.

In case that the foundation girder is designed as a non-hinged member, the moment developed at the pile top in accordance with the boundary condition of pile and foundation girder shall be taken into account.

8.2 Design of Foundation

Reliable strength shall be used in the design of structural members in the foundation.

[Commentary]

Reliable strength of the foundation structure should be defined so that an excessive deformation shall not develop in the foundation system. The design criteria shall be referred to the materials in the following: "Recommendations for Design of Building Foundations (revised in 1988)" [Ref. 8.2] and "Ultimate Strength and Ductility in the Seismic Design of Buildings" [Ref. 8.3] published by the A.I.J.

Reliable vertical strength of a footing foundation shall be defined considering allowable deformation. In case when liquefaction will is expected, adequate countermeasures shall be taken.

Reliable strength shall refer to an ultimate supporting capacity of piles. Evaluation of reliable strength of piles shall be also defined by an applicable settlement of piles that shall be reasonable to the super structure. Piles supporting the foundation of shear wall should be checked their safety against the overturning moment of the shear wall. The total yield mechanism by uplifting of a shear wall can be regarded as one of preferable yield mechanisms. In this mechanism, however, if the pull-out capacity of piles could be not correctly evaluated, the failure mechanism of the shear wall could change from the planned mechanism.

For evaluation of the pull-out resistance of piles, the following items shall be considered:

(1) tensile longitudinal strength of piles,
(2) capacity of the pile-footing joint,
(3) friction capacity between piles and soils, and
(4) capacity of the footing when pulling out of the pile occurs

Friction of the foundation base shall be included as the horizontal resistance of the foundation. The passive soil pressure might be counted for the resistance for the buried foundation. It is commonly known that buildings with a basement floor reveal better seismic performance than those without a basement. Passive soil pressure and side friction to foundation structure are examined in the reference [8.4]. This reference, however, discusses issues for the working stress design.

For the design of piles against horizontal forces assuming that pile-cap is fixed, the piles of which ductility capacities are interior shall not yield in flexure nor fail in shear. The piles whose ductility capacities are superior shall not develop a failure in shear after yielding in flexure.

Major horizontal resistance of piles will be based on both strength and rigidity of the top of the piles. Considering the damage of piles observed during the earthquakes [Ref. 8.5], the top zone of piles shall be adequately strengthened by both longitudinal and lateral reinforcing bars.

When the pile top joints and the footing are designed under the condition of free rotation, the joint of pile and footing shall be designed paying much attention to the detail of arrangement of reinforcing bars. There are some cases that the footing-column joints designed as a pin-connected joint suffered damage during an earthquake on the possible reason that they were actually worked as a fixed joint in construction.

Both flexural and shear capacities of a pile shall be calculated by the method introduced in Chapters 5 and 6. For the calculation of a steel jacked pile, the effect of corrosion of steel should be considered [Refs. 8.6 and 8.7].

8.3 Embedment Depth of Foundation

Foundation of a structure shall be placed with a sufficient embedment.

[Commentary]

Deep embedment of a foundation gives predominance of structural safety and reduction of horizontal forces to the foundation. It is often recommended that the embedment depth of a foundation shall be taken larger than 8 percent of the total height of the building. Further research and development shall be needed for further examination on the theme discussed hereby.

8.4 Design of Foundation Girders

> Foundation girders, as a general rule, shall be designed rigid

[Commentary]

A foundation girders shall be, in principle, designed as a non-hinge members It may be desirable that a yield hinge is formed at the end of a foundation girder to develop a beam sidesway yield mechanism On the other hand, however, considering the following two items described in the following, planning of yield hinges in the foundation girders is not recommendable: (1) The foundation girders are embedded under the ground, and (2) the foundation structure should support safely the super structure.

Therefore, yield hinges in the foundation girders are not desirable except the ends of the girders that are connecting to structural walls.

8.5 Design of Pile Cap

> A pile cap shall be designed carefully not to deteriorate the capacities of a pile and its connection to the foundation slab and girder

[Commentary]

Flexural moment and tensile axial force of a pile cap due to the horizontal load shall be transferred to the foundation slab and girder The pile caps shall be embedded fully into the foundation slab and a sufficient anchorage capacity shall be ensured It is considered that the horizontal force would be transmitted from the foundation slab to the pile cap by the dowel action at the cross section of embedment portion of the pile Structural details and reinforcing arrangement of a pile cap are described in the reference [8.5]

Further discussion will be required on the theme whether piles shall positively share horizontal loads or not. One can point out that the piles should support the vertical loads performing no horizontal capacities. In this case, less attention shall be paid for the damage of piles. However, one should establish other possible countermeasures to transfer the horizontal forces acted on the super structure to the supporting soil through the foundation structure, and should pay much attention on the connection of the pile caps and foundation slabs.

8.6 Liquefaction

> Possibility of liquefaction shall be examined through investigation on the supporting soil, and shall be taken into the design.

[Commentary]

In case that soil that supports the foundation consists of fine sand and is saturated, or is under the conditions equivalent mentioned above, it is required that the soil liquefaction shall be carefully investigated Sand layers that meet the following conditions will be subjected to liquefaction with a high probability

(1) less than 15–20 meters deep from the ground level, and the effective pressure is less than 20tonf/m^2;

(2) comparatively homogeneous, and of composed of medium size grains with the contamment ratio of silt and clay less than 10%,

(3) is located below the groundwater level, and is saturated with water; and

(4) relative density of sand less than 75%, and the N value obtained from the standard penetration test is relatively small.

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CHAPTER 9 . REINFORCEMENT DETAILING

9.1 Scope

Provisions of this Chapter shall apply for design of lateral reinforcement detailing, anchorage and splicing. Unless otherwise specified in this Chapter, general detailing requirements about bar sizes, bar spacing, concrete cover, and standard hooks shall conform to the "AIJ Standard for Structural Calculation of Reinforced Concrete Structures," the "Japanese Architectural Standard Specification for Reinforced Concrete Work, JASS-5," and the "Guidelines for Reinforcement Detailing" by Architectural Institute of Japan

[Commentary]

In the guidelines proposed, one reinforcement detailing is specified for the hinge end, and another for the non-hinge end. The detailing proposed herein will require higher performance than that specified in the conventional standards or guidelines, which requires greater amount of lateral reinforcing bars or more sophisticated placement of lateral bars.

The detailing for anchorage, development length of longitudinal bars are, as a general rule, examined based on an ultimate strength design procedure. Unless specified within this Chapter, general detailing shall be established in accordance with the Standards, Standard Specification, Guidelines and others published from the AIJ [Refs. 9.1–9.4]

9.2 Placement of Lateral Reinforcement

9.2.1 Lateral Reinforcement

Lateral reinforcement is placed as shear reinforcement and confining reinforcement. The amount, shape and spacing of reinforcement are separately specified for the yield hinge and non-yield hinge regions, respectively. Deformed bars of size greater than or equal to 10mm (D10) or deformed PC bars of nominal diameter greater than or equal to 6.4 mm shall be used as lateral reinforcement.

9.2.2 Shape, Arrangement and Spacing

Lateral reinforcement shall be made in shape and arrangement effective to confine concrete and longitudinal reinforcement. Maximum spacing shall satisfy the values listed in Table 9.1

TABLE 9.1 MAXIMUM SPACING OF LATERAL REINFORCEMENT (unit : cm)

Location	Type and Size	Hinge Region	Non-hinge Region*1
Column*2	Deformed bars D10*3	10	15
	Deformed bars larger than D10*4	15 and 6 d_b	20 and 8 d_b
Beam	Deformed Bars D10*3	15	20
	Deformed Bars larger than D10*4	20, 8 d_b and D/3	30, 10 d_b and D/2
Wall Panel	Deformed Bars D10 and larger	30	30
Beam-Column Connection	Deformed Bars D10*3	——	15
	Deformed Bars larger than D10*4	——	20 and 8 d_b

[Note] d_b: bar diameter; and D: beam depth.

- *1 including region outside of a yield hinge region
- *2 including boundary columns
- *3 including deformed PC bars larger than or equal to 6.4 mm and less than or equal to 9.2 mm in nominal diameter
- *4 including deformed PC bars larger than or equal to 11 mm in nominal diameter

9.2.3 Yield Hinge Region

Yield hinge region shall be as follows:

(1) in beams, region within 1.5 times beam depth from column face toward beam center,

(2) in columns, region within 1.5 times column depth from beam facetoward column center; and

(3) in structural walls, region within 1/6 of total height from the wall base or horizontal width, whichever larger, but not higher than the bottom of the third floor beams

[Commentary]

9.2.1 Lateral Reinforcement

(1) Definition of Lateral Reinforcement

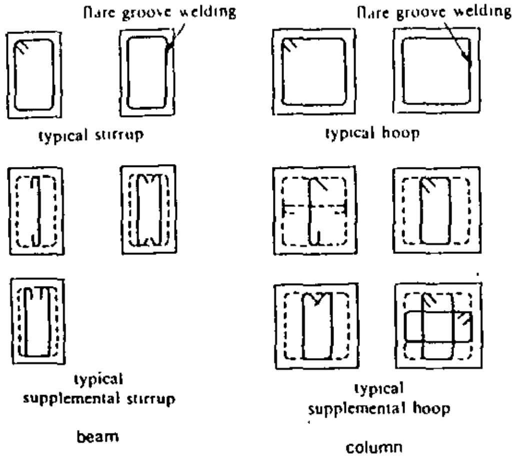
In Japan, the reinforcement arranged orthogonally to the longitudinal reinforcement is termed as "hoop" in columns, and "stirrup" in beams. In addition, auxiliary reinforcement provided for longitudinal reinforcement in the intermediate part of cross section is termed as "supplemental stirrup," "sub-tie," "sub-hoop" or "supplemental hoop." These reinforcements are provided mainly to resist shear force, and for this reason they are generally termed as "shear reinforcement." The "wall reinforcement" placed in walls is also one form of shear reinforcement. Various examples of hoops, ties and stirrups are shown in the "Guidelines for Reinforcement Detailing" published by the AIJ (Fig C9.1) In the guidelines the term "lateral reinforcement" as well as "shear reinforcement" is used

The lateral reinforcement placed orthogonally to the longitudinal reinforcement plays the following four roles:

i) to resist shear forces;
ii) to prevent buckling of the longitudinal reinforcement;
iii) to confine the core concrete; and
iv) to prevent bond splitting

However, it is not possible to calculate the amount necessary or to design the reinforcement configuration required for each of these roles The shear reinforcement calculated in accordance with Chapter 6 of the present guidelines, can also be expected to simultaneously perform the four roles above However, the effect of roles ii) – iv) on the member's deflection capacity will vary depending on the shape and arrangement of shear reinforcement even though the amount is the same. Additionally, it is necessary for members with high axial load to provide a large amount of lateral reinforcement to prevent buckling of longitudinal reinforcements and to confine the core concrete in order to achieve the required deflection capacity. In this case the roles ii) and iii) above become the most important.

This chapter mainly specifies the shape, spacing, and arrangement of reinforcement necessary to ensure the assurance deflection of members In other words the detailing for confining reinforcement which plays the role of shear reinforcement as well is specified. This type of multi-role reinforcement is termed "lateral reinforcement"



[Note] 180° hook may be replaced by 135° hook

Fig. C9.1 Shape and name of shear reinforcement.

(2) Basic Design Policy for Lateral Reinforcement

The basic design policy for lateral reinforcement in yield hinge region is to determine the necessary amount, spacing and arrangement of reinforcement to ensure the deflection capacity larger than the assurance deflection In the present design guidelines, the assurance deflection for buildings is taken as 1/66 for frame structures and 1/75 for shear wall structures in terms of deflection angle (see the Commentary section 4.4.5) Referring to experimental data, provisions of reinforcement detailing for hinge regions are given in this chapter for achieving target deflection capacity of members given below corresponding to the values above:

i) for columns and general beams · 1/50;
ii) for boundary beams connected to shear walls : 1/40; and
iii) for shear walls : 1/75.

For columns carrying high axial loads, or shear walls with boundary columns subjected to high axial loads, special considerations of lateral reinforcement are necessary so as to provide sufficient confinement effect in order to obtain the deflection capacity described above. In such members the hinge region is considered be special, and it must be dealt in a different manner from that of normal hinges. Also, for boundary beams connected to shear walls, while it is expected that these will sustain greater deformation than general beams, this effect has already been taken into account in the calculation of the required shear reinforcement introduced in Chapter 6, and the provisions of the Chapter 6 provide adequate assurance of deflection capacity without any special details Hence, hinges in these boundary beams are treated as normal hinges. Regarding the lateral reinforcement in members without yield hinges, or in non-hinge regions in members with yield hinges, it is sufficient to arrange the required lateral reinforcement provided with the Chapter 6 in accordance with the spacing provided in the section 9.2.2

9.2.2 Shape, Arrangement and Spacing of Lateral Reinforcement

(1) Shape and Arrangement of Lateral Reinforcement

In order to ensure the confining effect of lateral reinforcement provided in hinge regions to confine both the longitudinal reinforcement and concrete, it is essential to arrange the lateral reinforcement in a closed shape around the perimeter of the longitudinal bars confining corner bars and also to arrange lateral reinforcement to confine intermediate longitudinal bars within the section. By confining all the longitudinal bars, the best confining effect will be achieved. Lateral reinforcement in hinge regions shall be anchored to the longitudinal bars by means of a hook with shape sufficiently large angular bend, or shall be in a spiral shape. Alternatively, the ends of the bars may be welded to give a closed shape.

For lateral reinforcement in non-yield hinge regions of members with yield hinges, it is desirable that the shape and arrangement are corresponding to those used in hinge regions

(2) Spacing of Lateral Reinforcement

In order to achieve the confining effect in yield hinge regions with lateral reinforcement, an important factor together with the amount, shape, and arrangement is reinforcement spacing This chapter provides the required spacing of lateral reinforcement in hinge regions of beams and columns (including boundary columns in shear walls) from the viewpoint of preventing buckling of longitudinal reinforcement The required spacings are given in Table 9.1 corresponding to the diameter of longitudinal reinforcement. In non-hinge regions these provisions may be lightened.

The research project of reinforced concrete short columns promoted by the Ministry of Construction recommended shorter spacing than eight times the diameter of longitudinal reinforcement to prevent premature buckling of longitudinal reinforcement. On the other hand in the "AIJ Standard for Structural Calculation of Reinforced Concrete Structures" [Ref. 9.1], the spacing of hoop of 10mm (D10) size in the end regions of columns is given as 10cm or less. This would correspond to the space less than five times the longitudinal bar diameter of 19mm (D19). However, for hoop of the size of 13mm (D13) or larger, the required spacing is 20cm or less, and the ratio to the bar diameter may become very large in some cases.

In this chapter, while the recommended values above have been taken in reference, it was also noted that the present guidelines would be applied to columns under high axial load which were not covered in detail during the short column research project cited above. Taking the current recommendations of the "AIJ Standard for Structural Calculation of Reinforced Concrete Structures" into consideration, greater values than those in the above references were taken, providing a basic spacing of six times the longitudinal bar diameter or less. For beams a distinction is also made between yield hinge and non-yield hinge regions, however, a minimum spacing of eight times the longitudinal bar diameter is provided for hinge regions due to the absence of axial load.

9.2.3 Yield Hinge Regions

Hinge regions dealt with in this Chapter are defined as the regions in which the lateral reinforcement is differentiated from that in other regions and arranged accordingly. The length of hinge region along the member-axis is determined based on various proposals derived from the experimental research described below.

Yielding of longitudinal reinforcement in a member in which yield hinges are formed not only at the critical section of the end of member, but also within a certain length toward the center of the member. The length of hinge region is influenced by structural factors such as shear-span ratio, axial stress level, tensile reinforcement ratio, and amount of lateral reinforcement. Proposals have been made for the quantification of these effects by Yamada, Mattock [Ref 9.7], Yoshioka, and Park [Ref 9.8]. Yamada expresses the length as a function of the effective depth of the member and the location of neutral axis, and Mattock points out that the hinge length changes with the shear-span ratio. Yoshioka takes the hinge region as the interval in which bending-shear cracks are concentrated and proposes the functional relationship given in the following equation assuming the hinge length, l_p, changes with the shear-span ratio, (M/QD)

$$l_p = 0.5(M/QD)\ d$$
$$(0.5 \leqq M/QD \leqq 3.0)$$

where d designates the effective depth. It is noted that the hinge length does not increase proportionally to the shear-span ratio in the range of large shear-span ratio.

In Park's research the hinge length is expressed in terms of the shear-span M/Q and the longitudinal reinforcement diameter dp by the following equation

$$l_p = 0.08(M/Q)+6d_b$$

In the same research it is noted that the length l_p increases with span/depth ratio within the maximum value of about 4, and that on average l_p=0.5 D (D=column depth). Further, it is suggested that the l_p value may be reduced in accordance with the axial load ratio to half the value obtained from the above equation for the cases where the axial load ratio is less than or equal to 1/3. However, the influence of axial load is not remarkable.

As shown by these researches, the yield hinge region is at most about 1.5 times the effective depth for columns with shear-span ratio up to 3.0, and it is considered that it may be possible to employ an even shorter hinge length corresponding to shear-span ratio. However, because the experimental observations of hinge length have shown a wide scatter, and due to the limited amount of research on hinge length, it was decided to fix the hinge length for columns at 1.5 times the overall depth (D) regardless of shear-span ratio consisting with the column hoop provisions in the "AIJ Standard for Structural Calculation of Reinforced Concrete Structures". For small shear-span ratios this means that a longer hinge region is taken for safe side calculation. On the other hand, it is expected that hinge lengths greater than that specified above will occur as the shear-span ratio increases. However, in view of the small level of this increase, and the fact that the applied shear force will not be great for this type of column, it was considered that there would be no difficulty in achieving the required deflection capacity even though the region containing closely spaced lateral reinforcement may not be long. It was thus decided that a hinge length greater than that specified above may not be specified even where the shear-span ratio exceeds 3.0.

Beams generally have shear-span ratios greater than those of columns. However, it is considered that very slender beams will not often be designed because of the requirement to ensure necessary lateral resistance and stiffness of a building. In addition, it has been pointed out that the hinge length reaches a peak at a span/depth ratio of about 4 (shear-span ratio of 2 at anti-symmetric bending), and that the hinge length may be decreased as axial load decreases. For these reasons as well as the consideration above of columns with large shear-span ratios, the hinge region for beams is treated in the same way as for columns, having a length of 1.5 times the beam depth.

There has not been a great deal of research on hinge lengths in shear walls. However, according to the review of experimental results on multi-story shear walls, the diagonal cracking is concentrated, in most cases, in a region extending from the story at which yielding has occurred (generally the lowest story) into the wall of the story above by a height corresponding roughly to the horizontal length of the wall. The height of this region is influenced by the number of stories and the distribution of lateral forces, and also, there are examples of yielding of longitudinal reinforcement of boundary columns extending even to upper stories. It is reported in the same research, however, that the region of concentrated plastic deformation is shorter in walls than those of beams and columns due to the existence of beams or floor slabs, and it is unlikely that the yielding of longitudinal reinforcement of boundary column will extend higher than two stories. Considering these points, the hinge length has been determined in relation to the horizontal length and height of the wall within the region not extending over more than two stories.

9.3 Lateral Reinforcement in Special Yield Hinge Region

9.3.1 Definition of Special Yield Hinge Region

Special Yield hinge region is a yield hinge region of columns and structural walls in which axial loads in the yield mechanism assuring design fall in a range given by Eqs (9.1) and (9 2).

$$(1/3)\ Ac\ \sigma_B < N_c \leqq (2/3)\ A_c\ \sigma_B \qquad (9.1)$$

$$(2/3)\ A_{core}\ \sigma_B - A_{ws}\ \sigma_{ws} < N_w \leqq A_{core}\ \sigma_B - A_{ws}\ \sigma_{ws} \qquad (9.2)$$

where

N_c :axial load of column in the yield mechanism assuring design (positive in compression),

N_w :axial load of wall in the yield mechanism assuring design (positive in compression);

A_c :area of column horizontal section;

A_{core} :area of boundary column core on compression side of the wall;

A_{ws} :area of vertical reinforcement in wall panel,

σ_B :compressive strength of concrete, and

σ_{ws} :material strength of vertical reinforcement in wall panel for upper bound strength calculation

9.3.2 Arrangement of Lateral Reinforcement

In a special yield hinge region, all longitudinal reinforcement shall be supported by the corner of closed shape lateral reinforcement or supported by the hook with 135 degree or more of lateral reinforcement. A longitudinal reinforcement may not be confined if the bar is placed within 20 cm distance of supported longitudinal bars. Minimum spacing of lateral reinforcement shall satisfy the value for a yield hinge region in Table 9.1.

9.3.3 End of Lateral Reinforcement

End of lateral reinforcement placed on a special yield hinge region must conform to the followings:

(1) Ends of closed-shape lateral reinforcement shall be anchored with more than 135 degree hook having more than 8 d_w extension. The extension shall be more than 10 d_w for lateral reinforcement of deformed PC bars

(2) Ends of lateral reinforcement having other than closed shape shall be anchored with more than 135 degree bend, and 4 d_w for a 180 degree bend, where d_w denotes the diameter of lateral reinforcement.

[Commentary]

9.3 1 Definition of Special Yield Hinge Regions

(1) Regions Covered as Special Hinge Regions

Special hinge regions are the hinge regions where special considerations as to the design of lateral reinforcement are required to achieve the required deflection capacity. Such regions include those of exterior columns, for example, which carry high axial compression, or those of shear walls with boundary columns subjected to high axial compression.

(2) The Level of Axial Compression of Columns and deflection capacity

The reduction in deflection capacity of columns with increase of axial load has been observed in many experimental studies In the test by Higashi et al., for example, it is reported that the upper limit of axial load ratio to assure the deflection capacity of 2/100 or more is about 0 3. In addition there have been several review studies on the relationship between axial load and deflection capacity using available test data. On the other hand, under the provisions for classification of member ductility in the present Building Standard Law, the columns with axial load ratio greater than 0 35 are classified into the ranks FB - FD inferior to the highest ductility rank FA . Looking at the results of a series of short column tests which were used as the back data for the classification of member ductility, all the columns of "ductility rank A", having the deflection capacity of 6 or more in terms of the ductility ratio and presumably corresponding to the FA rank members, achieved the assurance deflection angle of more than 1/50. Lower ductility rank columns often sustained the limit deflection angles smaller than 1/50.

From the examples above it is considered that special considerations for lateral reinforcement are necessary to ensure assurance deflection for the cases where axial load ratio exceeds 1/3. This aspect was investigated by studying the tests of over 100 columns subject to high axial compression. The ranges of structural parameters of the test columns were.

i) column cross section at least 25cm square,
ii) axial load ratio 0 29 – 0.78,
iii) shear-span ratio 1.1 – 3.5;
iv) concrete strength 206 –558 kgf/cm^2;
v) lateral reinforcement ratio 0 44 – 1 71%; and
vi) yield strength of lateral reinforcement 3000 – 14000 kgf/cm^2.

In the majority of the test specimens lateral reinforcements are arranged so as to provide the confinement for the intermediate longitudinal bars as well as corner bars. However, other several specimens do not provide such confining detailing to the intermediate reinforcement. The level of shear stress at the maximum load was between 7 - 27% of the concrete strength in terms of the nominal stress to the total cross-sectional area of the column Lateral force were applied to form a hinge at each end, using the system of anti-symmetric loading. Many of the columns had relatively short shear-span ratios. There were hardly any data where hinges were formed only at the column bottoms, as assumed in the present guidelines, and shear-span ratios were greater than those obtained from anti-symmetric bending. Specimens subjected to alternate tension and compression axial loads, as expected in exterior columns, were included.

Figure C9 2 shows the relationship between limit deflection angle (Ru) and axial load ratio ($N_c/(A_c\ \sigma_B)$) for the columns in these experiments The limit deflection angle is taken as the deflection angle where 80% of the load carrying capacity can be maintained after the maximum load The relationship between axial load ratio and limit deflection angle is not well correlated, however, it was clear that most the observed limit deflection angles were smaller than 1/50 for the axial load ratio greater than 1/3. In particular, the deflection capacity was less than 1/50 for the majority cases where the intermediate longitudinal reinforcement was not confined (marked with a symbol of + in the figure). In contrast, for axial load ratios less than this, all the test columns showed deflection capabilities larger than 1/50, with the exception of one case having no intermediate bar confinement.

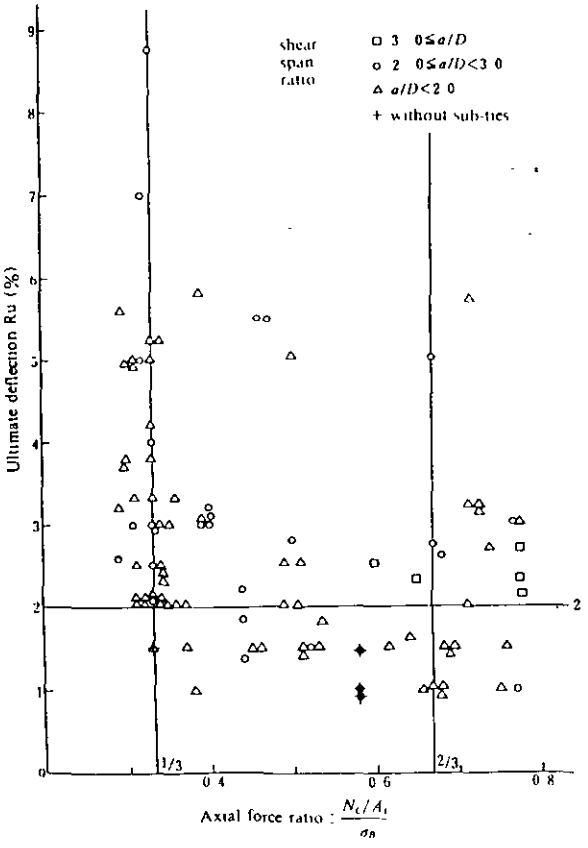


Fig. C9.2 Axial force level and deflection capacity of columns in test data.

For the columns, such as exterior columns, where high axial compression forces act during an earthquake, the axial load ratio can easily exceed 1/3 However, as shown in the following investigations, the assurance deflection can be ensured if the lateral reinforcement evaluated in Chapter 6 is appropriately arranged Therefore the axial load ratio over 1/3 is permitted in the present guidelines The upper limit of the axial load ratio is set as 2/3 referring to the data in Fig. C9 2, the results of bending analysis of the column section mentioned in Chapter 5, and design examples of high-rise reinforced concrete buildings.

For shear walls it is also confirmed from the experiments that the deflection capacity reduces with increase of axial compression. However, in comparison with columns, the data showing this effect are remarkably few. According to the analysis based on various experimental results of flexural failure type shear walls, the relationship between nominal axial compression of boundary column and the limit deflection, is shown as Fig 5 18 The nominal value was determined assuming that all the compressive axial load on the shear wall under the ultimate strength is carried by the boundary column on compression side From the figure and the conversion of axial load ratio to that to the cross section of core concrete of a boundary column, it was considered be difficult to ensure the deflection capacity beyond 1/75 for the cases of nominal axial compression larger than 2/3. Where the nominal value was determined to the sectional area of core concrete instead of whole sectional area, of a boundary columns In consideration of this, special hinge regions are provided for the cases with nominal axial compression to the core concrete beyond 2/3. This is the same boundary condition for an boundary column where the ratio of the sectional area of core concrete and the total sectional area is 2/3. Referring to the same figure and the upper limit for columns, an upper bound was set at 1.0 to the core concrete of a boundary column.

9 3 2 Arrangement of Lateral Reinforcement

If the amount of lateral reinforcement meets that of shear reinforcement evaluated in Chapter 6, basically greater deflection capacity in hinge regions than the assurance deflections can be assured. This is confirmed in the investigations described below even for the case of columns subjected to high axial load. However, it is also noted in the following investigation that it is necessary to provide reinforcement to laterally confine the intermediate longitudinal bars to ensure the required deflection capacity. For this reason it is required that the lateral reinforcement be arranged in accordance with this section in addition to satisfaction of Chapter 6.

However, the deflection capacity of columns subject to high bending and extremely high axial load has not been sufficiently investigated because of the scarcity of experimental data It is considered that there may be cases where the amount of reinforcement obtained from Chapter 6 does not provide sufficient confinement, and for these cases, the provisions of this section shall be applied appropriately increasing the amount required in Chapter 6

(1) Shear Design Equations and deflection capacity of Columns Subject to High Axial Load.

In Fig. C9.3 the relationship between the limit deflection angle and the ratio of required amount of lateral reinforcement by the design equation (6 1) to the amount of actually arranged reinforcement, is shown for the column specimens used in the investigation shown in Fig C9 2 The required amount of lateral reinforcement was calculated as that to assure the deflection angle of 1/50 The input shear force for the equation was taken as the maximum shear force obtained during the test for each column.

In the figure, although there are many data where the amount of reinforcement is less than the value evaluated with Eq (6.1), among them there are a significant number which show the deflection angles large than the target value of 1/50 In the cases where more amount of reinforcement than that required by the equation is provided, all the columns showed larger deflection capabilities than the target value with the exception of columns where the intermediate longitudinal bars were not confined

As indicated by the above comparison, if Eq (6 1) is satisfied and the intermediate longitudinal bars are laterally confined, deflection capacity of 1/50 can be ensured Thus, it was decided for columns with axial load ratios between 1/3 and 2/3 that the arrangement of lateral reinforcement shall follow the provisions of this section in addition to satisfying Eq (6 1), providing confinement to the intermediate longitudinal reinforcement In addition, while there is little experimental data showing the relationship between axial load level of a boundary column of shear wall, it was decided that the arrangement of reinforcement shall follow the recommendations of this section as well as fulfilling the requirements of Chapter 6

(2) Arrangement of Lateral Reinforcement

In principle, the amount and arrangement of lateral reinforcement in columns or shear wall boundary columns subject to high axial load should be determined considering the factors such as the strength and stiffness of lateral reinforcement, the buckling length of longitudinal reinforcement, and the effective cross sectional area of the concrete core. Although a study considering these factors has been tried by Kato, in practice however, it is difficult to make quantitative assessments With regard to the arrangement of lateral reinforcement within a section, it is recommended in the "Ultimate Strength and Deformation Capacity of Buildings in Seismic Design" that rectangular or spiral hoops should be provided at a spacing not more than 10cm within the length at least 1.5 times the effective column depth away from the connecting faces of beams or other members In addition, auxiliary hoops should be arranged to confine at least half the number of intermediate longitudinal reinforcement to confine the concrete and any intermediate longitudinal reinforcement (excluding corner reinforcements) It also recommends that the intermediate longitudinal reinforcement be confined at the interval about 20cm. The reason for the recommended spacing of 20cm is not stated. However, it is thought that the value was set considering that while closely spaced lateral reinforcements would achieve high confining effect, they may obstruct the concrete placing work. It can also be considered that the spacing of the intermediate longitudinal bars requiring confinement could be changed depending on the cross sectional dimension of columns. In the NZS standards [Ref. 9.5], it is specified that all the longitudinal reinforcement shall be confined using lateral reinforcement for the cases where the spacing between intermediate longitudinal bars exceeds 20cm However, the numerical basis for the use of this value is not made clear

Due to the absence of any established methods for determining the required amount and arrangement of confining reinforcement, the present section specifies that where the spacing between longitudinal bars is 20cm or more, all such bars shall be confined using lateral reinforcement arranged at the spacings given in Table 9.1 taking into account the recommended values such as those above.

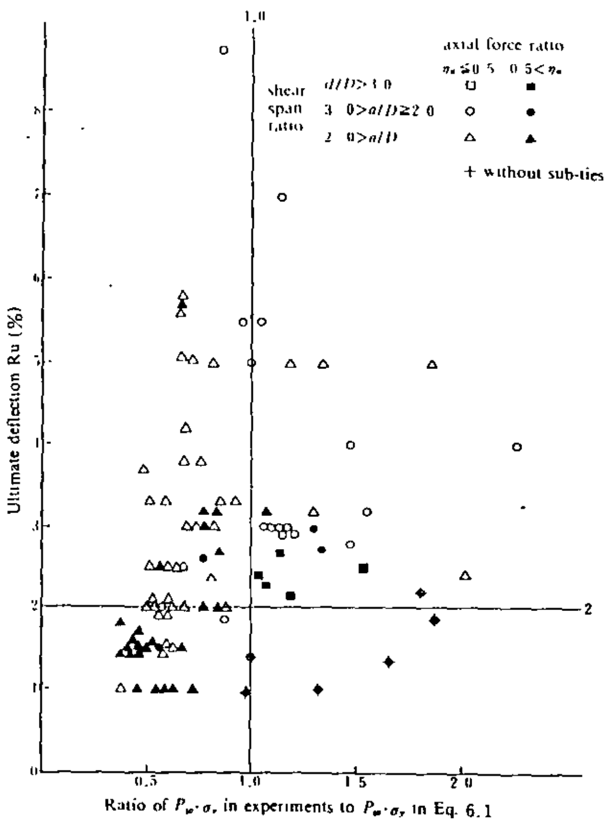
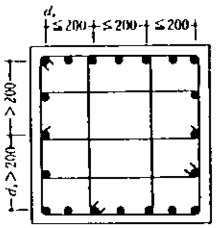


Fig. C9.3 Amount of lateral reinforcement and deflection capacity of columns in test data.



d_r : spacing between every second bars

Fig. C9.4 Arrangement of lateral reinforcement in the special hinge region.

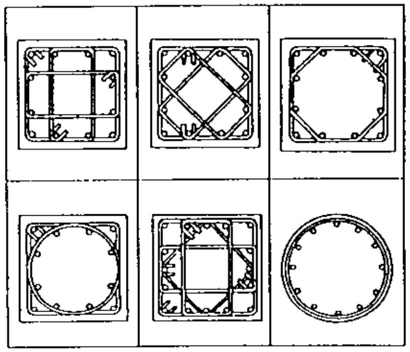


Fig. C9.5 Examples of recommended arrangement of lateral reinforcement.

9.4 Anchorage and Splice of Longitudinal Reinforcement

Development length and splice length of longitudinal reinforcement shall be of sufficient value for design actions in the yield mechanism assuring design. Reinforcement detailing shall conform to the requirements of the "AIJ Standard for Structural Calculation of Reinforced Concrete Structures," the "Japanese Architectural Standard Specification for Reinforced Concrete Work, JASS-5" and the "Guidelines for Reinforcement Detailing" published by Architectural Institute of Japan. Lap splice shall not be used for reinforcing bars of size greater than or equal to D29.

[Commentary]

(1) Development Length of Longitudinal Reinforcement

Where longitudinal reinforcement of beams or columns is anchored by bend within beam-column joint, it is necessary to ensure an adequate length against the stresses developed at the yield mechanism. The required length is basically dependent on fundamental parameters such as diameter and strength of longitudinal reinforcement and the concrete strength. In addition, it is also dependent on other parameters such as thickness of cover concrete, reinforcement spacing, and the amount and arrangement of lateral reinforcement. However, because of the lack of experimental data relating to the strength and development length, and the lack of systematic analysis of any data available, it is currently difficult to establish a rational calculation method for the required development length taking various factors given above into consideration.

For this reason this chapter makes only the recommendations described below for the minimum development length referring to the current standards, "AIJ Standard for Structural Calculation of Reinforced Concrete Structures", "Guidelines for Reinforcement Detailing" and other standards from foreign countries, and experimental results in Japan.

i) The overall development length shall be that as shown for normal use length in the "AIJ Standard for Structural Calculation of Reinforced Concrete Structures," the "Guidelines for Reinforcement Detailing," and "JASS-5"

ii) The vertical length (development tail length) shall be 12 times the reinforcement diameter.

iii) The horizontal projected length l_{dh} shall be determined as follows, taking a length of 10 times the reinforcement diameter as a base for a combination of SD30 reinforcement and Fc360 concrete

$$l_{dh} = 10d_b(\sigma_y /3000) \cdot \sqrt{(360/Fc)}$$

where σ_y is the minimum specified yield strength of the reinforcement.

The minimum development length above is recommended based on the following discussions:

i) The difference in the development length calculated in accordance with the ultimate strength design methods in foreign countries and the length specified as the normal use length based on the allowable stress design method used in Japan is not remarkable. Therefore, it is considered that the longitudinal reinforcements will be sufficiently anchored against the stresses development during the upper bound strength defined in these guidelines when this normal use length is ensured.

ii) Available experimental data indicate that the yield strength of reinforcement can be achieved and the deflection capacity larger than the assurance deflection is obtained when the horizontal length above is ensured.

In Japan, many experimental studies have been carried out on the anchorage strength and development length of reinforcement. In this section the test results have been reviewed paying attention to the relationship between anchorage strength or deflection capacity and the horizontal projected length of reinforcement. In the tests the loading pattern was either (1) monotonic uni-directional tensile loading directly on reinforcements anchored within a column using a 90 degree bend, or (2) monotonic or cyclic lateral loading to a model of exterior beam-to-column joint with beam longitudinal reinforcement anchored within the column using 90 degree bend. The range of bar diameters used in the tests was D10–D25 (mainly D16 or D19) with a yield strength of approximately 3300–4000 kgf/cm^2, and the range of concrete compressive strength 220–420 kgf/cm^2. The interior radius of the bend anchorage was 2–4 d_b. The data of beams where the bottom reinforcement anchored by bending down was in tension were omitted.

Figure C9.6 shows the relationship between anchorage strength and the horizontal projected length of longitudinal reinforcement, and Fig. C9.7 shows the relationship between deflection capacity (limit deflection angle) and horizontally projected length of longitudinal reinforcement. Figure C9.6 indicates that the yielding can be developed when the horizontal projected length is 10 or more times the longitudinal bar diameter. Figure C9.7 indicates that the deflection capacity larger than 1/50 can be attained when the horizontal projected length is 10 times the longitudinal bar diameter or longer.

Looking at the distribution of material strength, it is safe side evaluation to consider that the above results indicate the required horizontal development length for a combination of 360kgf/cm2 concrete, which is the upper limit of the scope of the present guidelines, and SD30 or SD35 class reinforcement. For the combination of concrete and reinforcement with another strength, it is sufficient to revise the required length in accordance with Eq. (9.3) of this commentary. Regarding the extension, the provisions above were made referring to the values recommended in the "AIJ Standard for Structural Calculation of Reinforced Concrete Structures (1988)" and to foreign standards due to the scarcity of quantitative data.

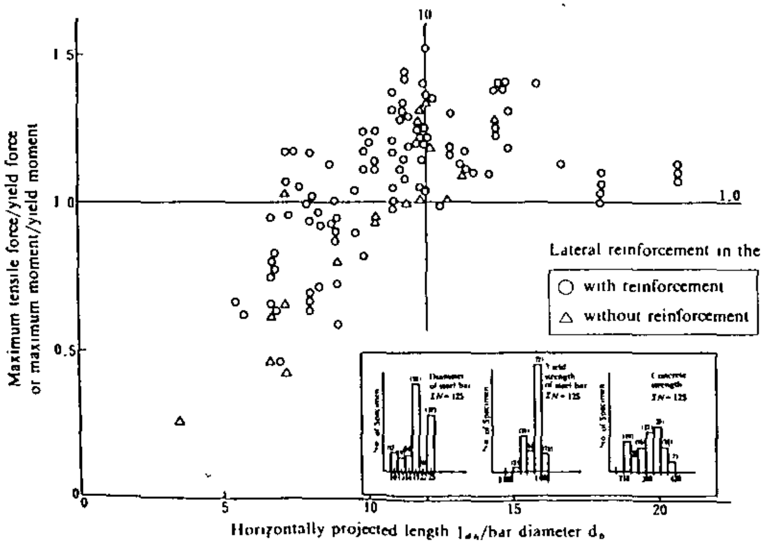


Fig. C9.6 Anchorage strength and horizontally projected length.

(2) Splice Length of Reinforcement

Where reinforcements are spliced by lapping, bond splitting is an important problem. The required lap length shall be decided after the check of bond splitting in accordance with the provisions of Chapter 6. Alternatively, the minimum lap length may be taken as the normal use length specified in the "AIJ Standard for Structural Calculation of Reinforced Concrete", "JASS-5", or the "Guidelines for Reinforcement Detailing".

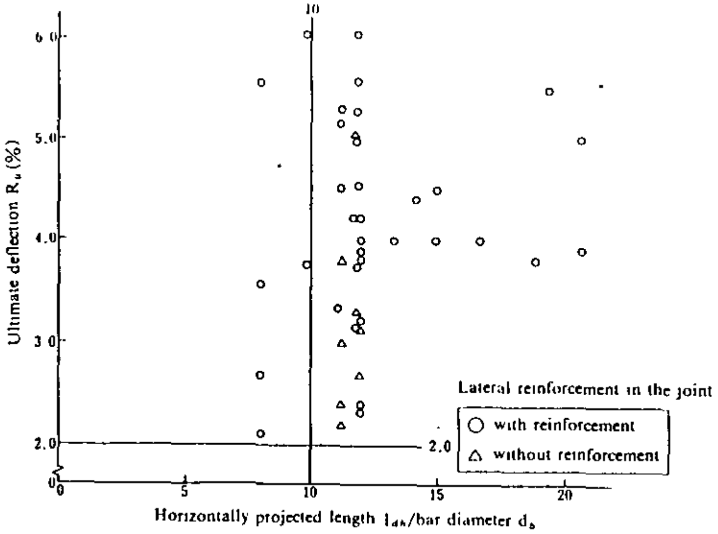


Fig. C9.7 Deflection capacity and horizontally projected length.

NOTES AND ACKNOWLEDGEMENTS

A large number of research papers written in Japanese, which have been referred to in the original Japanese edition, are not compiled in this English edition It should be noted that the contents of this chapter cannot be established unless the great contributions of studies conducted by a large number of Japanese researchers are utilized.

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CHAPTER 10 NON-STRUCTURAL ELEMENTS

10.1 Scope

Provisions of Chapter 10 shall apply for design of structural detailing of reinforced concrete non-structural elements placed within a beam-column frame. The provisions may be used for non-structural elements other than the reinforced concrete construction.

[Commentary]

The details of the joint between structural members and reinforced concrete non-structural elements should be designed by the provisions of this chapter, in order to plan the desirable seismic behavior in frames and to ensure it The provisions of this chapter may be applied for the jointing design of non-structural elements other than cast-in-concrete such as the partitions or exterior walls constructed by bricks, concrete blocks or precast concrete panels inside or outside of a frame.

10 2 Design Principles

Reinforced concrete non-structural elements, as a general rule, shall be separated from a structure by placing gaps, and shall be designed not to interfere with the structural behavior expected in both the yield mechanism design and the yield mechanism assuring design.

[Commentary]

Reinforced concrete spandrels or wing walls are often constructed monolithically with a beam-column frame in building construction in Japan. They are usually designed as non-structural members in the structural design. Damages during earthquakes, however, reveal the lesson that the reinforced concrete spandrels or wing walls which are often assumed as non-structural elements have disturbed the assumed structural behavior of the frames in the structural design, and then the behavior of the non-structural elements caused the unexpected damage of structural members.

The formation of flexural yield hinges is generally planned at the bottom of a column at the first story and both the ends of a beam at each story in the structural design based on this guideline The reinforced concrete spandrel or wing walls constructed monolithically with a column at the first story may interfere the formation of yield hinge at the bottom of the column. Also the spandrels or wing walls may interfere the formation of yield hinges at the beam ends of more than the second story. The non-structural elements cast concrete simultaneously with the structural beams and columns should be effectively separated by placing gaps between the structural and non-structural elements to ensure the flexural yield mechanism planned for a frame The non-structural walls at the midspan of a beam also should be separated from the beam by gaps

When the spandrels or wing walls are designed as structural members, all the provisions given in Chapters 5, 6, 7 and 9 shall be required for them The design technique to construct a reinforced concrete spandrel or wing wall as a non-structural element without gaps has not been developed yet

The failure of non-structural elements may cause overturning or falling of the non-structural element itself, and also the failure may affect the function of non-structural element itself even if it may not affect the structural members The small reinforced concrete non-structural walls, which have supported the entrance doors of each residence in a high rise residential building, cracked widely by the lateral deformation of the beam-column frame during the 1978 Miyagiken-oki earthquake. The failure of the non-structural walls affected opening the doors, and then the occupant could not get out of their residences immediately after the earthquake

The design to prevent the failure or falling of non-structural element itself due to the inertia force or deformation caused by a beam-column frame during an earthquake should comply with another guidelines issued by the AIJ [Ref. 10.1]

10.3 Separation Between Structural and Non-structural Elements

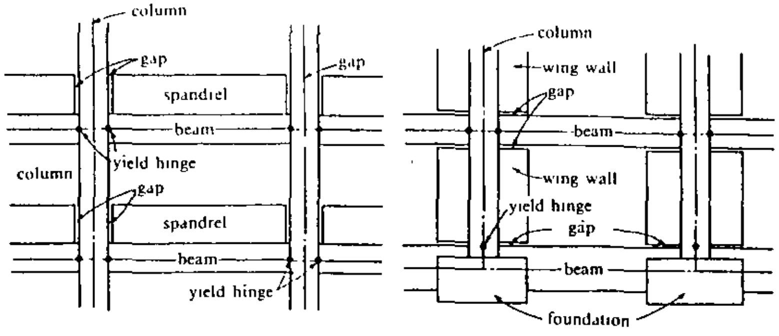
(1) If non-structural elements such as spandrels or wing walls are to be constructed at yield hinge region planned in the yield mechanism design, gaps shall be placed between the structural and the non-structural elements at the critical section of beams or columns

(2) If non-structural elements are to be placed in mid-span, gaps shall be placed along top or bottom edges of the non-structural elements to ensure the formation of a yield mechanism planned in the yield mechanism design

[Commentary]

(1) The locations to separate spandrels or wing walls from a frame are recommended in Figs C10.1(a) and (b) Gaps should be placed at all the location between the columns and the spandrels as shown in Fig. C10.1(a). Also gaps should be placed at all the location between the beams and the wing walls as shown in Fig. C10.1(b). The flexural yield hinges are generally planned at the beam ends in each floor level and the bottom of the columns in the first floor level. The gap should be placed at the same location as a flexural yield hinge planned.

(2) The location to separate non-structural wall at the midspan of a beam is recommended in Fig. C10.2. This kind of a non-structural wall which is sometimes constructed in the residential buildings may induce the unexpected additional shear force in the beam by the truss action of the non-structural wall, so the wall should be separated from the frame to prevent the shear failure of the beam and to ensure the flexural yield mechanism planned



(a) Spandrel walls (b) Wing walls.

Fig. C 10.1 Location for separation (type-A).

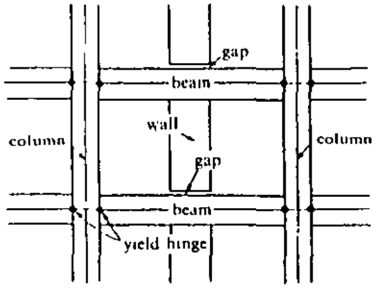


Fig. C 10.2 Location for separation (type-B).

10.4 Details for Separation

(1) Available gap shall be either complete separation type or shear failure type.

(2) Minimum gap width d_s shall be given by Eq (10.1):

$$ds = Rs\ hs\ l/L \qquad (10.1)$$

where Rs : assuring story drift angle;

hs : height of spandrels or width of wing walls;

l : span length of beams or height of columns; and

L : clear span of beams or clear height of columns.

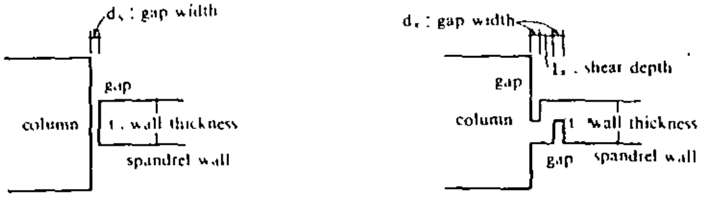
[Commentary]

(1) In the structural design guidelines for reinforced concrete structures by the Japan Building Center [Ref 10 2], it is required that the thickness at the joint between a non-structural reinforced

concrete spandrel or wing wall and a column or a beam should be less than 10 cm The several details about the joint between structural and non-structural elements have been developed to satisfy the requirement in the guidelines, and some details have been tested to investigate the structural performance in the laboratories.

The relationships between the lateral load and displacement by the tests of the model with the gaps of complete separation type or the shear failure type are shown in Figs. C10.5(a) and (b) [Refs. 10.11 and 10.12]. The test results indicate that the gap of shear failure type as shown in Fig C10 4 did not affect the structural behavior of the frame designed as the beam-yielding type, and the gap of shear failure type was effective as well as the complete separation type.

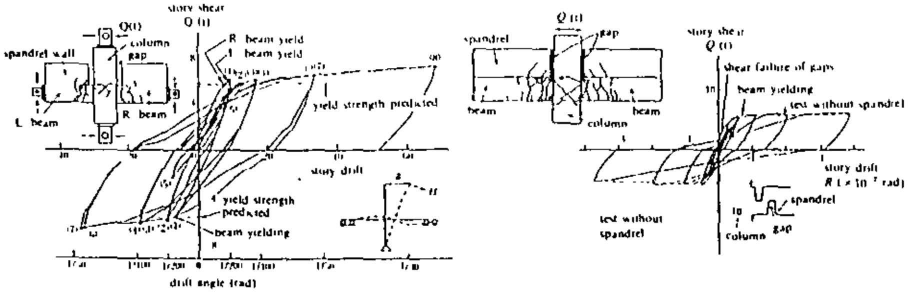
The shear depth l_s should be less than 0.5 times the wall thickness and less than 5 cm for the gap of a shear failure type. Any other details with the remaining concrete thickness at the gap should be tested and discussed about the effectiveness



(a) Perfect separation type.

(b) Shear failure type.

Fig. C10.4 Available gaps.



(a) Test with perfect separation type gaps
[Ref. 10.11]

(b) Test with shear failure type gaps
[Ref. 10.12].

Fig. C 10.5 Tests of beam-column sub-assmblages with gaps.

(2) The enough gap clearance should be provided to prevent the closing of the gap during an earthquake. Equation 10 1 for the required minimum clearance is derived from an assumption in which a beam or column rotates as a rigid body at the member's end as shown in Fig C10 6. The validity on Eq. 10.1 is examined by the empirical results [Refs 10.11 and 10.12].

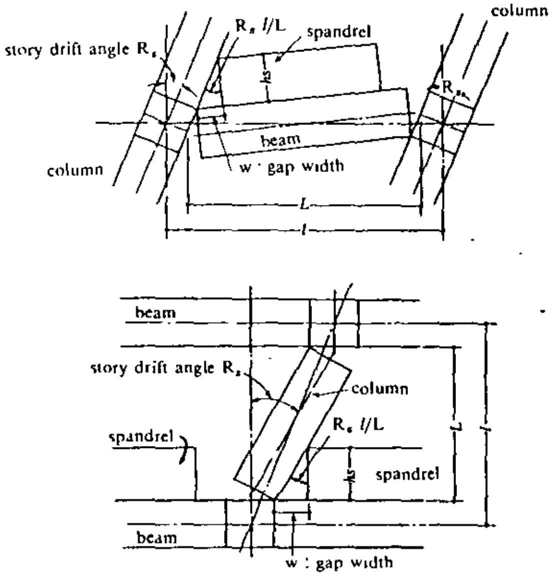


Fig. C 10.6 Assumptions of the required gap width.

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FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

XXVI CURSO INTERNACIONAL DE INGENIERIA SÍSMICA

MOUDLO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

REQUISITOS DE DUCTIBILIDAD PARA ESTRUCTURAS DE CONCRETO, DISEÑO SISMICO DE ESTRUCTURAS DE CONCRETO

EXPOSITOR: DR. OSCAR LOPEZ BATIZ
PALACIO DE MINERIA
SEPTIEMBRE DEL 2000

GUIA DE ESTUDIO

1. Desempeño en sismos recientes
2. Comportamiento del concreto ante cargas alternadas
3. Comportamiento de estructuras hiperestáticas
4. Criterios de diseño sísmico
5. Sistemas estructurales
6. Marcos dúctiles
7. Losas planas
8. Muros de concreto
9. Referencias.

GUIA DE ESTUDIO

1. Desempeño en sismos recientes

El número de construcciones dañadas ha sido elevado. Principalmente defectos de estructuración, falta de ductilidad.

El sismo de 85 en México puso en evidencia toda la gama de problemas.

Evidencia de buen comportamiento cuando se han seguido las prácticas adecuadas.

Las modificaciones al Reglamento y Normas Técnicas han sido muy fuertes. Las estructuras han cambiado radicalmente después de 85.

2. Comportamiento del concreto ante cargas alternadas

Para estructuras que deben resistir efectos sísmicos se requiere un comportamiento dúctil ante cargas laterales y un comportamiento estable ante repeticiones de carga alternadas. Los ciclos de histeresis deben contener un área grande para que la estructura pueda disipar energía mediante amortiguamiento inelástico.

El concreto simple es un material frágil, tanto en tensión como en compresión.

Variables que influyen en la curva esfuerzo-deformación: velocidad de carga, f'_c ,

Estado biaxial y triaxial de esfuerzos y el efecto del comportamiento.

El confinamiento con zuncho o con una combinación de estribos y barras longitudinales es el único medio de lograr un comportamiento dúctil.

El comportamineto deseable se puede lograr sólo cuando el modo de falla que domina es el de flexión o flexo compresión con carga axial muy baja.

La sección debe ser ampliamente subreforzada, doblemente armada.

Se requiere confinamiento en las articulaciones plásticas y especialmente evitar el pandeo del refuerzo en compresión.

Las fallas por flexocompresión, cortante, torsión, adherencia no garantizan comportamiento dúctil.

3. Comportamiento de estructuras hiperestáticas

Los criterios de diseño sísmico actuales se basan en la consideración de que sólo parte de la energía del sismo se disipa por trabajo de la estructura en su intervalo de comportamiento lineal.

Para sismos excepcionales se tendrán deformaciones inelásticas y hay que dar a la estructura capacidad para entrar en esa etapa sin daño grave o colapso.

El concreto tiene comportamiento no lineal desde niveles de carga moderados: agrietamiento por flexión.

El comportamiento netamente no lineal se tiene con la fluencia de secciones por momento flexionante.

El comportamiento no lineal implica redistribución de momentos. Las secciones que se agrietan o fluyen pierden rigidez o se articulan y aumentan los momentos en las zonas que permanecen más rígidas.

La viga continua representa un ejemplo simple del fenómeno de redistribución.

En cada sección el momento actuante está limitado por el momento resistente (positivo y negativo) que dispone la sección de acuerdo con el refuerzo proporcionado.

Los momentos se redistribuyen de acuerdo a la resistencia disponible hasta que se forme un mecanismo de falla.

El mecanismo de falla que se presentará puede ser seleccionado en la etapa de diseño al definir los momentos resistentes de las distintas secciones.

Es aceptable dimensionar las secciones a partir de los diagramas de elementos mecánicos que se obtiene del análisis elásticos lineales. Idealmente en esa condición se alcanza simultáneamente la capacidad de todas las secciones. También es factible dimensionar para elementos mecánicos diferentes de las elásticas y que cumplan con el equilibrio.

Los criterios de diseño de las normas actuales exigen diseñar de manera que se presenten mecanismos de falla dúctiles y tomar factores de seguridad adicionales para modos de falla frágiles o que correspondan a un comportamiento con deterioro.

4. Criterios de diseño sísmico

Esto se busca mediante el manejo de factores de resistencia diferentes o mediante la revisión de condiciones de equilibrio local (de nudo, de entrepiso, de viga o de columna), ver ejemplos.

Por estas condiciones el dimensionamiento se aleja mucho de los resultados del análisis elástico.

Los códigos permiten reducciones a los coeficientes sísmicos dependiendo de qué tan severos son los requisitos que se observan para garantizar un comportamiento dúctil.

Los códigos establecen requisitos de rigidez y de resistencia. Los primeros (desplazamientos admisibles) definen esencialmente las dimensiones de los elementos estructurales, los segundos el refuerzo. La distribución del refuerzo longitudinal y transversal obedece la búsqueda de los mecanismos de falla dúctiles.

Las reducciones por ductilidad de los coeficientes sísmicos deben limitarse para evitar daños frecuentes y reparaciones costosas.

5. Sistemas estructurales

La elección del sistema estructural apropiado es el paso básico del diseño.

Además debe evitar comportamientos indeseables por concentración de fuerzas, amplificaciones, vibraciones torsionales, etc.

Por ello debe buscarse un sistema regular y simétrico.

También se deben evitar concentraciones de fuerzas en la cimentación.

El marco "rígido" es un sistema relativamente flexible con el que resulta difícil limitar los desplazamientos laterales a los valores admisibles en edificios de cierta altura.

Conviene recurrir a rigidización de los marcos con muros de concreto de otros elementos.

6. Marcos dúctiles

La estructuración a base de marcos permite alcanzar grandes ductilidades. Para ello deben obedecerse requisitos estrictos de diseño y detallado de las vigas, columnas y conexiones viga-columna.

Los requisitos del RDF87 y del ACI son similares a este respecto. Sus objetivos son que las articulaciones plásticas se presenten en zonas especialmente detalladas para alcanzar grandes ductilidades y que aún las secciones donde se esperen articulaciones plásticas se protejan contra falla frágil.

Resumen de requisitos para vigas:

- Cuantía máxima de refuerzo igual a 50% de la balanceada.
- Tener un refuerzo mínimo positivo y negativo en todas las secciones ($P_{MIN} = 14/f_Y$); mínimo dos barras en cada lecho.
- Colocar en los extremos refuerzo positivo que proporcione un momento resistente igual por lo menos a la mitad del negativo.

- Por lo menos una tercera parte del refuerzo negativo debe extenderse hasta un cuarto del claro y una cuarta parte debe ser continua en todo el lecho superior.
- No cortar refuerzo en zonas de posibles articulaciones plásticas (a 2d del apoyo); si no pueden evitarse traslapes deberán colocarse estribos a lo largo de los mismos.
- Estribos, mínimo #3, a d/2 en toda la viga y a d/4 en una distancia de 4 peraltes a partir del apoyo. En esta zona $A, \geq 0.15A'_s \frac{s}{d}$ ó $0.15A_s \frac{s}{d}$.
- En la zona de articulación plástica (2d del apoyo) las barras que deban trabajar en compresión deberán estar confinadas por estribos (mínimo #3) a una separación no mayor de 16 Ø ni 30 cm.
- Debe diseñarse para la fuerza cortante que se presenta en la viga cuando se alcanzan los momentos últimos en los extremos. Esto es con la finalidad de que pueda desarrollarse un mecanismo de falla por flexión.

Resumen de requisitos para columnas:

- Cuantía de refuerzo entre 1 y 6%
- La suma de las capacidades en flexión de las columnas que concurren a una unión debe ser mayor que la suma de capacidades de las vigas que concurren a la misma. Esto tiende a asegurar que las articulaciones plásticas se formen en las vigas. No dice cuánto deben sobrediseñarse las columnas.
- Si $P \leq 0.4\ P_B$ (carga axial para falla balanceada) deben respetarse en la columna los mismos requisitos que para vigas.
- Cuando $P > 0.4\ P_B$ hay que confinar el núcleo de la columna por medio de espiral o estribos en una distancia igual a un peralte, 1/6 de la altura de la columna o 45 cm (el mayor de los tres) a partir de la carga de la viga.

- La cuantía de refuerzo espiral será

$$\rho_S = 0.45 \left(\frac{A_G}{A_C} - 1 \right) \frac{F_C}{F_Y} \leq 0.12 \ \frac{F'_C}{F_Y}$$

- El área de estribos de confinamiento será por lo menos igual a $A_{SH} = \frac{\ell_H \ {}_S S_H}{Z}$; S_H no mayor que 10 cm.
- Para reducir la longitud ℓ_H pueden emplearse ganchos del mismo diámetro que los estribos cuya deformación requiere retringir.
- Separación máxima de estribos: $d/2$; diseñados para resistir el cortante que se introduce en la columna al formarse las articulaciones plásticas en las vigas.

Las conexiones viga-columna son puntos críticos del comportamiento de un marco. Ha habido fallas frecuentes sobre todo por anclaje inadecuadas del refuerzo de las vigas.

Las conexiones extremas son mucho más críticas que las interiores.

Se requiere revisar las conexiones:

a) Por confinamiento prolongando el refuerzo transversal a los extremos de la columna, dentro de la conexión con la trabe.

b) Por cortante, revisando la conexión para una condición de cortante última.

c) Por anclaje; evitando traslapes; dando longitud de anclaje suficiente a las barras longitudinales (esto rige el tamaño de la columna), dando un tamaño suficiente a la viga y a la columna para permitir la inversión de esfuerzos.

7. Losas planas

Al no tener vigas francas se limita el efecto de marco; Resultan sistemas muy flexibles y con problemas de cortante en la conexión losa-columna.

GRAN NÚMERO DE FALLAS OBSERVADAS EN ESTE SISTEMA.

ES NECESARIO QUE TENGAN OTROS ELEMENTOS QUE TOMEN CARGAS LATERALES (MUROS).

EL REGLAMENTO DEL DISTRITO FEDERAL DA REQUISITOS DE ANÁLISIS (ANCHO EQUIVALENTE DE LOSA) Y DE DIMENSIONAMIENTO DE REFUERZO EN LOSA Y EN LA CONEXIÓN, PARA QUE RESISTAN EFECTOS SÍSMICOS. LA EFICIENCIA ES LIMITADA.

EL REFUERZO POR SISMO DEBE CONCENTRARSE EN LAS NERVADURAS DE EJE DE COLUMNA Y DEBE PROPORCIONARSE REFUERZO DE CORTANTE EN UNA VIGA AHOGADA.

8. MUROS DE CONCRETO

MAL LLAMADOS MUROS DE CORTANTE, TRABAJAN PRINCIPALMENTE POR FLEXIÓN.

PUEDEN ALCANZAR GRAN DUCTILIDAD SI SE DETALLAN APROPIADAMENTE. POR SU ALTA RIGIDEZ TIENDEN A CONCENTRAR LAS FUERZAS SÍSMICAS.

REQUIEREN REFUERZO VERTICAL Y HORIZONTAL EN EL ALMA Y, ESPECIALMENTE, REFUERZO EN SUS EXTREMOS PARA QUE CUANDO TRABAJEN ESTAS EN COMPRESIÓN NO SE PRODUZCA FALLA FRÁGIL.

LOS ELEMENTOS EXTREMOS DEL MURO DEBEN DETALLARSE COMO COLUMNAS DÚCTILES.

LOS HUECOS Y ABERTURAS REQUIEREN DE DETALLADO ESPECIAL.

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REQUISITOS DE DUCTILIDAD

PARA ESTRUCTURAS DE CONCRETO

ROBERTO MELI

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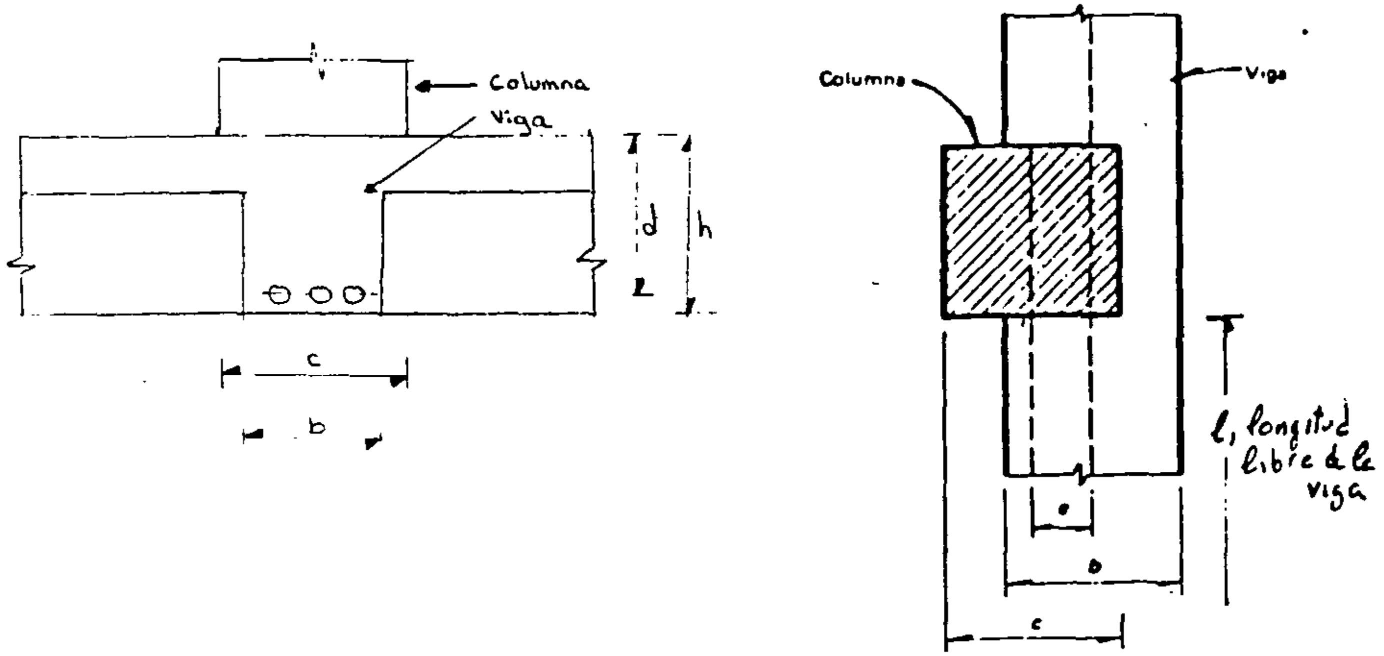


a) Requisitos Generales

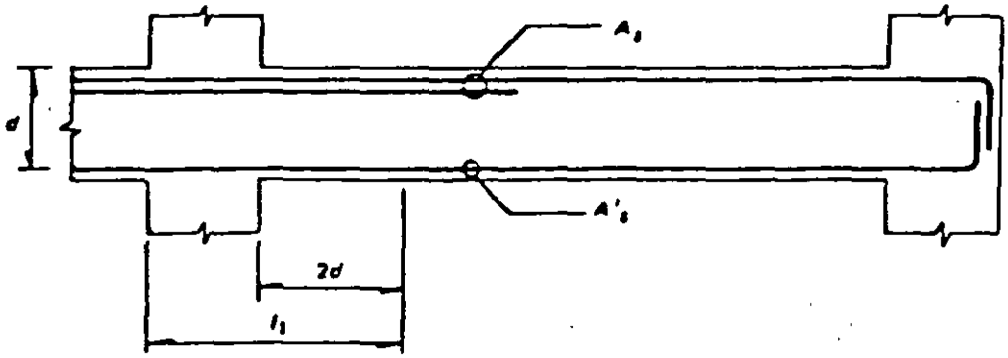
$\ell/b \leq 35$

b) Requisitos para marcos dúctiles

$b \geq 25\,cm$ $\quad b \leq c$ $\quad h/b \leq 3$

$e \leq 0.1c$ $\quad \ell/b \leq 30$ $\quad \ell/d \geq 4$

Fig 10.1 Requisitos geométricos para vigas de marcos de concreto.



a) Requisitos generales

A_s y $A'_s \geq 0.7 \frac{\sqrt{f'c}}{f_y} bd$, en zonas donde aparezcan tensiones

$A_s, A'_s \leq 0.75 A_{sb}$ (área de refuerzo correspondiente a falla balancead

b) Requisitos para marcos dúctiles

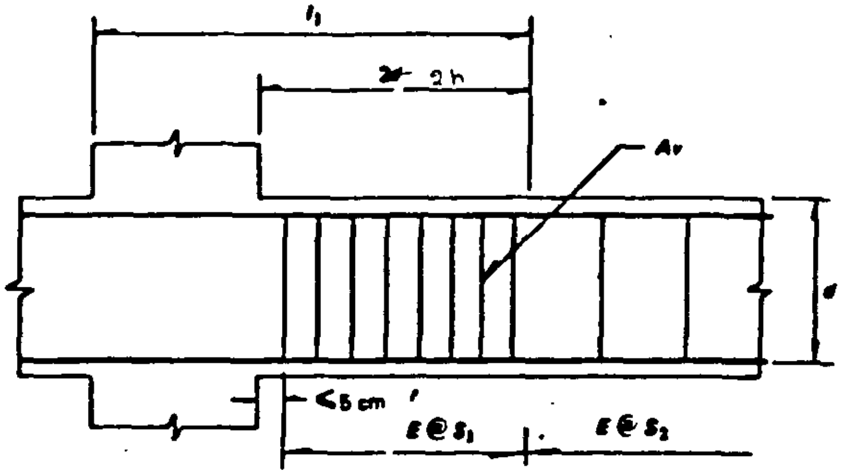
$A_s, A'_s \geq 0.7(\sqrt{f'c}/f_y) bd$, en toda la longitud de la viga
$A_s, A'_s \leq 0.75 A_{sb}$
Mínimo dos barras #4 en toda la longitud y en ambos lechos
No se admiten paquetes de más de dos barras
El momento resistente positivo en l_1 no será menor que la mitad del momento resistente negativo.

No puede haber traslapes, ni corte del refuerzo longitudinal en ℓ_1

Todo el refuerzo de tensión, A_s, necesario por sismo deberá pasar por el núcleo de la columna

En toda sección de la viga deberá proporcionarse una resistencia a momento negativo y positivo no menor que una cuarta parte de la máxima que se tiene en los extremos de la viga

Fig 10.2 Requisitos para el refuerzo longitudinal de vigas de marcos de concreto.



a) Requisitos generales

No se tendrá en cuenta estribos:
- que formen un ángulo con el eje de la pieza < 45°
- Ni barras dobladas en que dicho ángulo sea < 30°

Suministrar refuerzo mínimo por tensión diagonal cuando

si $V_u < V_{CR}$, $\phi \geq \#2$ @ $0.5d$ (Se colocará a partir de toda unión de viga con columna ó muro hasta 0.25 del claro correspondiente).

si $V_u > V_{CR}$, $S = \dfrac{F_R A_v f_y d (\operatorname{Sen}\theta + \operatorname{Cos}\theta)}{V_u - V_{CR}} \leq \dfrac{F_R A_v f_y}{3.5 b}$

NOTA: en secciones circulares se cambia d por el diámetro.

$$S \geq 5 \text{ cm}$$

$V_{CR} \leq V_u \leq 1.5 F_R b d \sqrt{f^*_c}$ $\qquad S \leq 0.5d$

$1.5 F_R b d \sqrt{f^*_c} < V_u$ $\qquad S \leq 0.25d$

En ningún caso se permitirá que $V_u > 2 F_R b d \sqrt{f^*_c}$

Cuando el refuerzo consiste de 1 solo estribo ó grupo de barras paralelas dobladas en una misma sección, su área se calculará $A_v = \dfrac{V_u - V_{CR}}{F_R f_y \operatorname{Sen}\theta}$, en éste caso no se admitirá $V_u > 1.5 F_R b d \sqrt{f^*_c}$

b) Requisitos para marcos dúctiles

1) Refuerzo Transversal para confinamiento

Estribos # 2.5 ó mayores

En la zona L_1 los estribos deberán ser cerrados y con remate a 135° se indica en la fig. 10.4. La separación no deberá exceder de:

$$S_1 \leq \begin{cases} 8 \text{ diámetros de la barra longitudinal mayor} \\ 24 \text{ diámetros del estribo} \\ 30 \text{ cm.} \\ d/4 \end{cases}$$

Ademas al menos una de cada dos barras longitudinales de la periferia deberá estar abrazada por la esquina de un estribo

2) Requisitos por cortante.

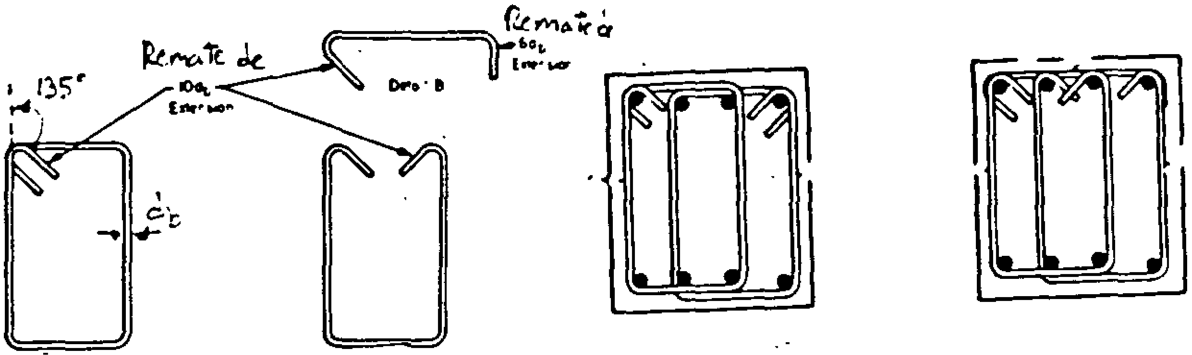
Fuera de L_1 habrá estribos a una separación $S_2 \leq d/2$

Estribos verticales cerrados de una pieza.

$\varnothing_E \geq 2.5$

$V_{sismo} \geq \frac{V_u}{2} \rightarrow V_{CR} = 0$

Fig. 10.3 Requisitos para refuerzo transversal de vigas de marcos de concreto



a) Estribo cerrado con remate a 135°

b) Estribo cerrado de dos piezas

c) Estribos cerrados dobles

d) Estribos cerrados con pieza de remate

Fig 10.4 Estribos para confinamiento (de código ACI)

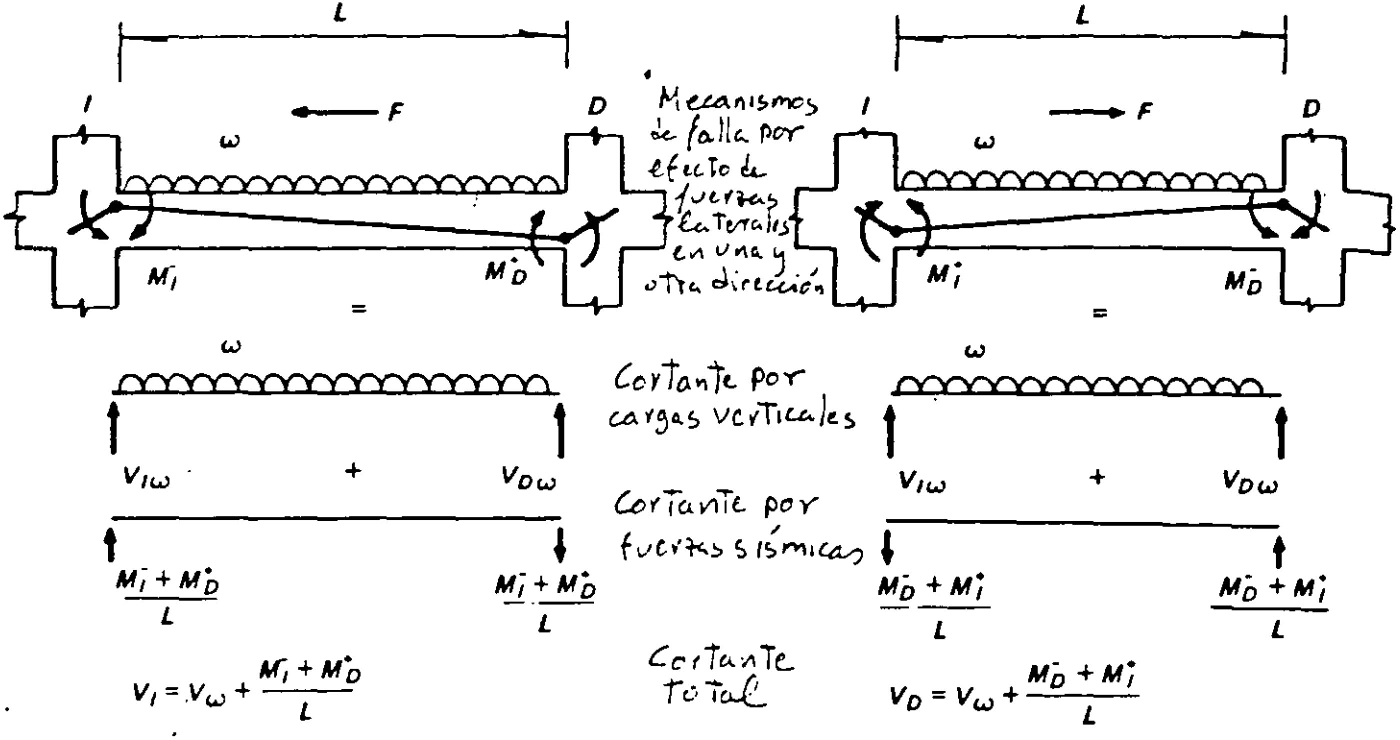
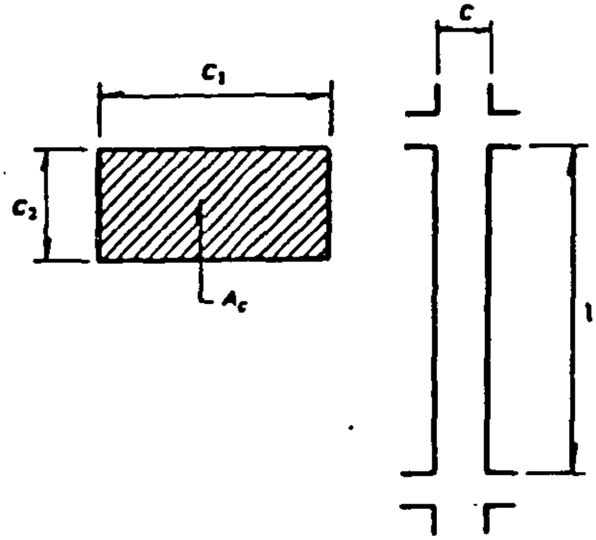


Fig 10.5 Determinación de las fuerzas cortantes de diseño para vigas de marcos dúctiles de concreto



Requisitos generales

$c_1, c_2 \geq 20$ cm

$c_1/c_2 \leq 4$

Requisitos adicionales para marcos dúctiles

$c_1, c_2 \geq 30$ cm

$A_c \geq \frac{P_u}{0.5 f'_c}$

$c_1/c_2 \leq 2.5$

$\ell/c \leq 15$

Fig 10.6 Requisitos geométricos para columnas de marcos de concreto

I.-REFUERZO LONGITUDINAL

a) Requisitos generales.

$$\frac{20}{f_y} \leq \rho \leq 0.06$$

Mínimo 4 barras en columnas rectangulares
y 6 en circulares

b) Requisitos para marcos dúctiles

$$0.01 \leq \rho \leq 0.04$$

Paquetes de no más de dos barras
Traslapes solo en la mitad central
de la altura libre de la columna

II.- REFUERZO TRANSVERSAL

El refuerzo transversal de toda columna no será menor que el necesario por resistencia a fuerza cortante y torsión, en su caso.

a) Requisitos generales

$$S_2 \leq \begin{cases} \dfrac{850\, d_b}{\sqrt{f_y}} \\ 48\, d_v \\ c_1/2\,,\ c_2/2 \end{cases}$$

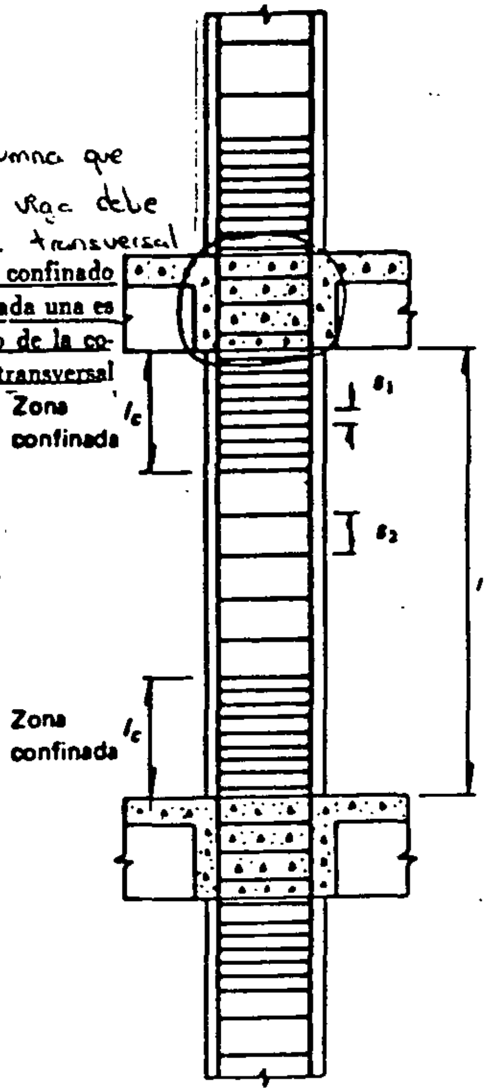
d_b - ∅ más delgado
d_v - ∅ estribo.

$S_1 \leq$ la mitad de los límites para S_2.

b) Requisitos para marcos dúctiles

1) Confinamiento

En la porción de la columna que atraviesa la unión con la viga debe colocarse el mismo ref. transversal que en l_c. Si el nudo está confinado por cuatro trabes que llegan a él y el ancho de cada una es al menos igual a 0.75 veces el ancho respectivo de la columna, puede usarse la mitad del refuerzo transversal mínimo.



$$l_c > \begin{cases} l/6 \\ 60\text{ cm} \\ C_1, C_2 \text{ (ver figura } \cancel{9.5}\ 10.6) \end{cases}$$

En la parte inferior de columnas de planta baja este refuerzo debe llegar hasta media altura de la columna, y debe continuarse dentro de la cimentación al menos en una distancia igual a la longitud de desarrollo en compresión de la barra más gruesa

El refuerzo transversal no debe ser de grado mayor que 42.
$d_v \geq \#3$
Refuerzo Transversal mínimo:

$$S_1 \leq \begin{cases} 10 \text{ cm.} \\ \frac{C_{menor}}{4} \end{cases}$$

S_1 — Mismos límites que para el caso general.

S_2 - Igual que para requisitos generales

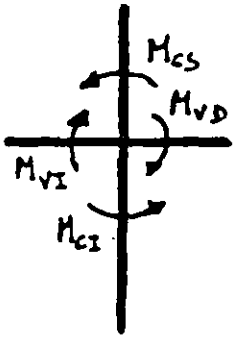
2) Fuerza cortante.

Si $P_u < \frac{A_g f'_c}{20}$ (Incluyendo efectos sismo)

$$V_{sismo} \geq \frac{V_u}{2} \rightarrow V_{CR} = 0$$

Fig. 10.7 Requisitos de refuerzo para columnas
de marcos de concreto.

$$\Sigma M_c \geq 1.5(\Sigma M_v)$$

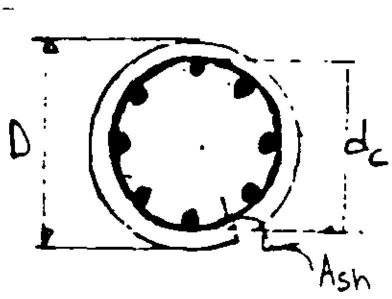


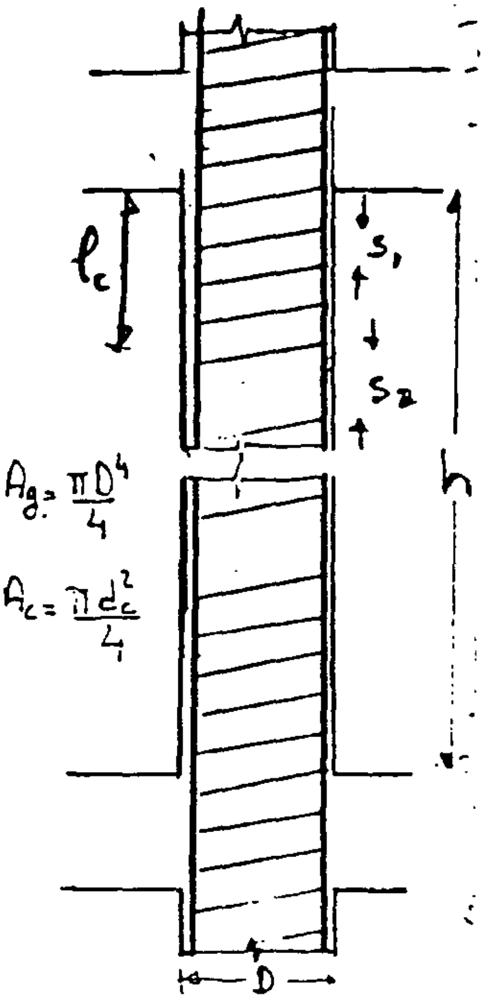
$\Sigma M_V = M_{VD} + M_{VI}$, es la suma de los momentos flexionantes resistentes (negativo de un lado y positivo del otro) de los extremos de las vigas que llegan a un nudo

$\Sigma M_c = M_{cs} + M_{cI}$, es la suma de los momentos flexionantes que deben ser capaces de resistir los extremos de las columnas (superior e inferior) que llegan a dicho nudo

El momento resistente de la columna se calculará para la carga axial que le corresponde a la columna por efecto de carga vertical más el doble de la que se genera por efecto de las fuerzas sísmicas actuando en la dirección correspondiente al signo de los momentos flexionantes considerados

Fig 10.8 Procedimiento para la revisión de la capacidad en flexocompresión de columnas de marcos dúctiles de concreto





$A_g = \frac{\pi D^4}{4}$

$A_c = \frac{\pi d_c^2}{4}$

Mínimo 6 barras longitudinales
Ref. Transversal Ø ≥ No. 3
El acero de la hélice ≤ grado 42

$$Ash \geq 0.11 \left(\frac{Ag}{Ac} - 1 \right) \frac{f'c}{fy} d_c S$$

$$Ash \geq 0.12 \frac{f'c}{fy} d_c S$$

$$l_c \geq \begin{cases} D \\ h/6 \\ 60 \text{ cm} \end{cases}$$

$S_2 \leq 7$ cm y ≥ 1.5 tamaño agregado

Los traslapes tendrán 1.5 vueltas
Las hélices se anclarán en los extremos de la columna mediante 2.5 vueltas

Fig **10.9** Requisitos para columnas zunchadas

$$\Sigma A_v \geq 0.3 \left(\frac{A_g}{A_c} - 1 \right) \frac{f'_c}{f_y} d_c S_1$$

$$\Sigma A_v \geq 0.12 \frac{f'_c}{f_y} d_c S_1$$

ΣA_v = suma de áreas de todas las ramas de estribos en la dirección considerada

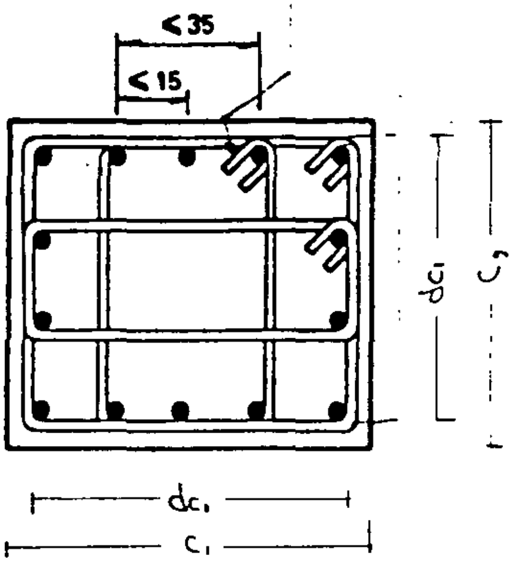


Fig. 10.10 Requisitos de distribución de refuerzo en columnas de estribos.

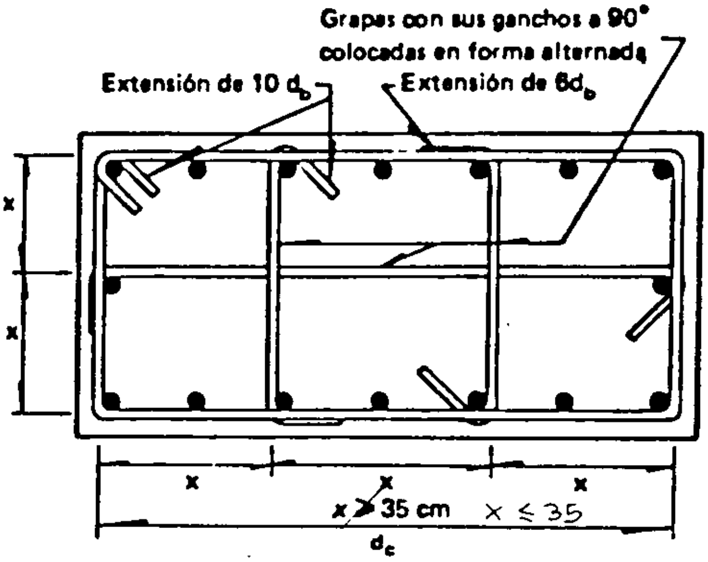
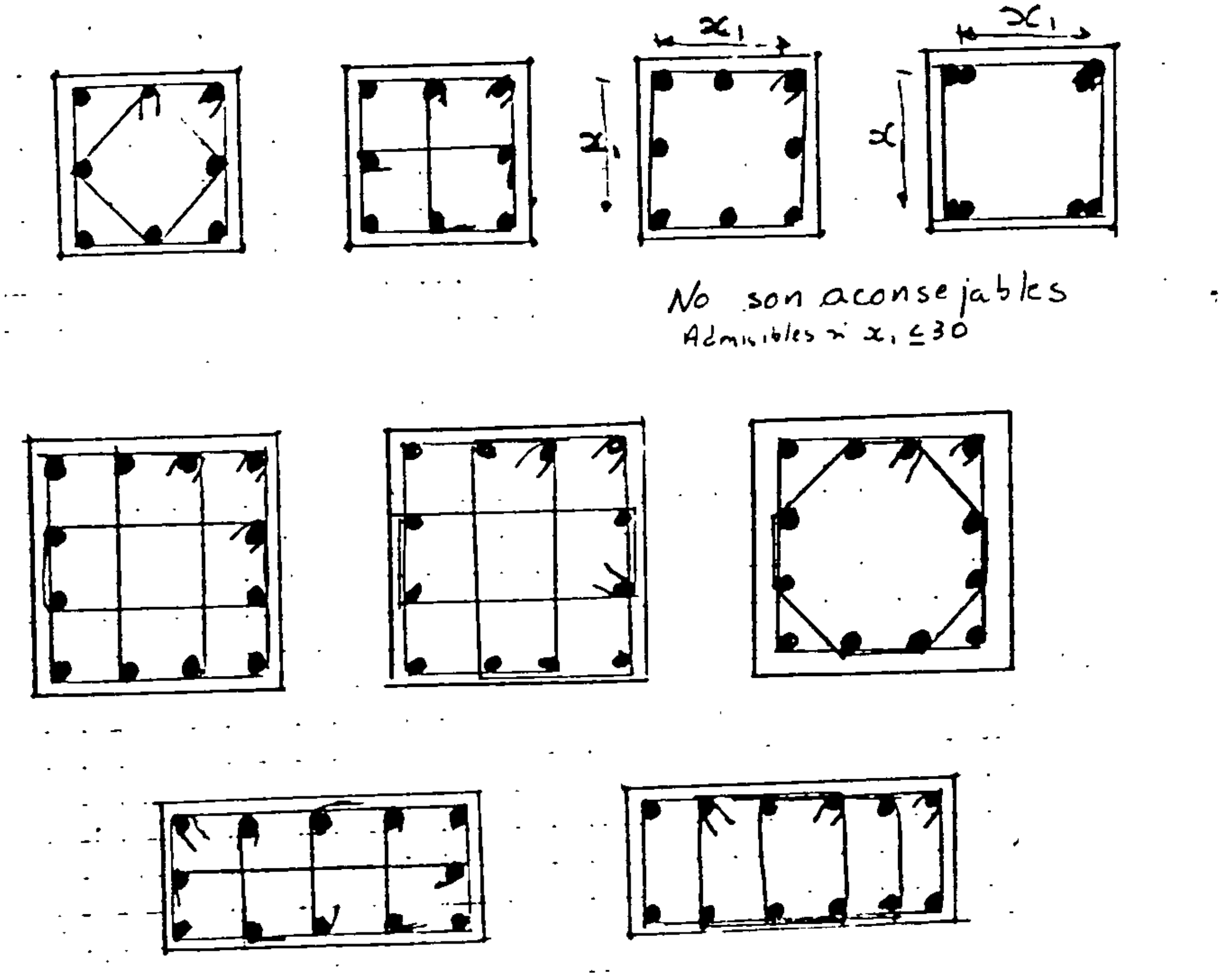
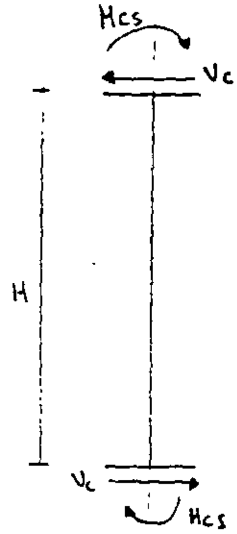


Figura ~~9.9~~ 10.11 Combinaciones de estribos y grapas admisibles para confinamiento de columnas, según el Reglamento ACI 83.



No son aconsejables
Admisibles si $x_1 \leq 30$

Fig 10.12 Arreglos admisibles de refuerzo en columnas de marcos dúctiles de concreto

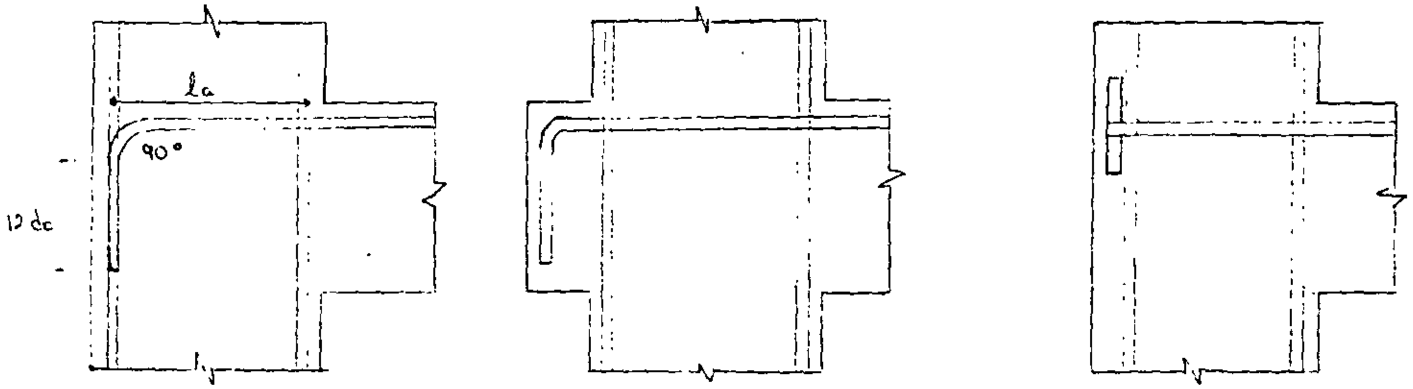


$$V_c = \frac{M_{cs} + M_{cs}}{H}$$

$$M_c \geq 1.5(\Sigma M_u)$$

- En el extremo inferior de columnas de planta baja, se usará el momento resistente de diseño de la columna obtenido con la carga axial de diseño que conduzca al mayor momento resistente.
- En el extremo superior de columnas del último entrepiso se usará $1.5 \Sigma M_u$.

Fig. 10.14 Procedimiento para la revisión de la capacidad por Cortante de las columnas de marcos dúctiles.



$$l_a \geq \begin{cases} \dfrac{0.076 f_y d_b}{\sqrt{f'_c}} \\ 15 \text{ cm} \\ 8 d_b \end{cases}$$

NOTA: 30 % mayor en concreto ligero

a) Anclaje con gancho extremo

b) Viga con saliente para proporcionar la longitud de anclaje necesario

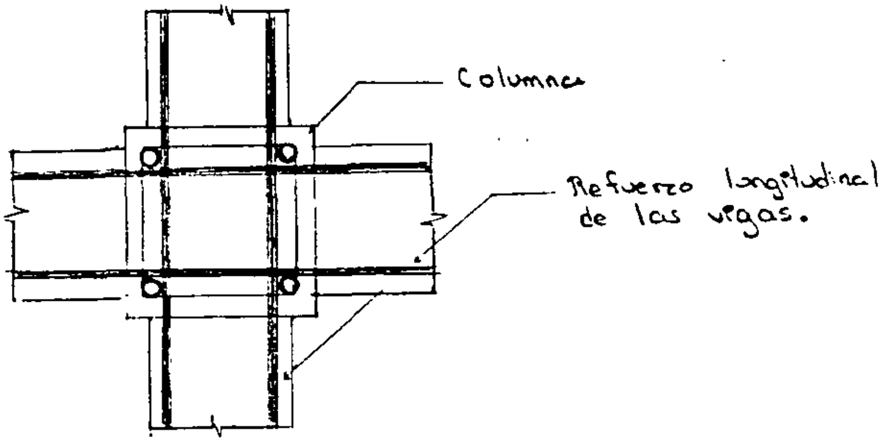
c) Anclaje mecánico

Fig. 10.15 Anclaje del refuerzo longitudinal en vigas extremas de

UNION VIGA- COLUMNA

Marcos Dúctiles

a) Requisitos generales

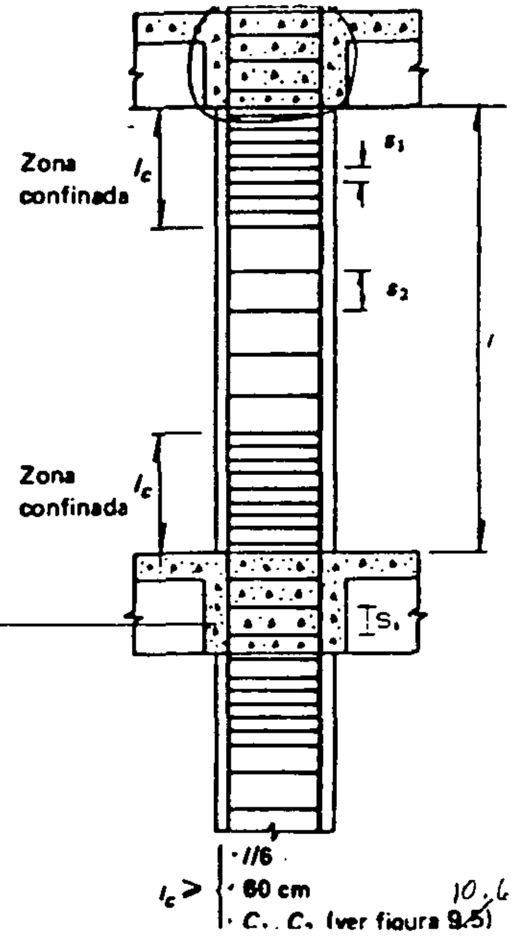


El refuerzo longitudinal de las vigas que llegan a la unión deve pasar dentro del núcleo de la columna.

b) Refuerzo Transversal

- El refuerzo transversal no debe ser de grado mayor que el 42.
- Estribos cerrados de una pieza. de $\phi \geq$ No.3
- Puede complementarse con grapas del mismo ϕ del estribo.

$$S_1 \leq \left| \begin{array}{l} 0.25\, C_{min} \\ 10\ cm \end{array} \right.$$



Si el nucleo está confinado por 4 trabes que llegan a él y el ancho de cada una es al menos igual a 0.75 veces el ancho respectivo de la columna, puede usarse la mitad del ref. transversal.

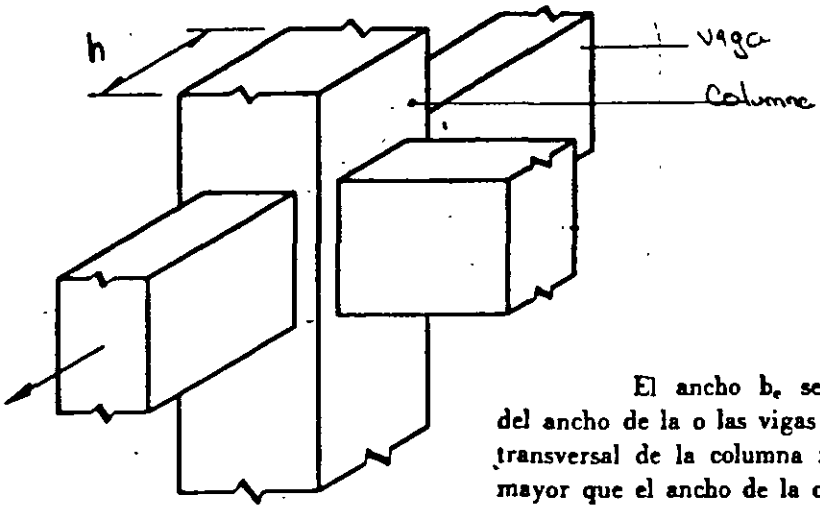
10.16 a)

c) Resistencia a fuerza cortante.

La fuerza cortante se calculará en un plano horizontal a media altura del nudo.

En nudos confinados $V_u = 5.5\, F_R \sqrt{f^*_c}\, b_e h$

otros nudos $V_u = 4.5\, F_R \sqrt{f^*_c}\, b_e h$



El ancho b_e se tomará igual al promedio del ancho de la o las vigas consideradas y la dimensión transversal de la columna normal a la fuerza, pero no mayor que el ancho de la o las vigas más h.

d) Anclaje del refuerzo

Los diámetros de las barras de vigas y columnas que pasen rectos a través de un nudo deben seleccionarse de modo que se cumplan las relaciones siguientes:

$h(col.)/d_b(\text{barra de viga}) \geq 20$
$h(viga)/d_b(\text{barra de col.}) \geq 20$

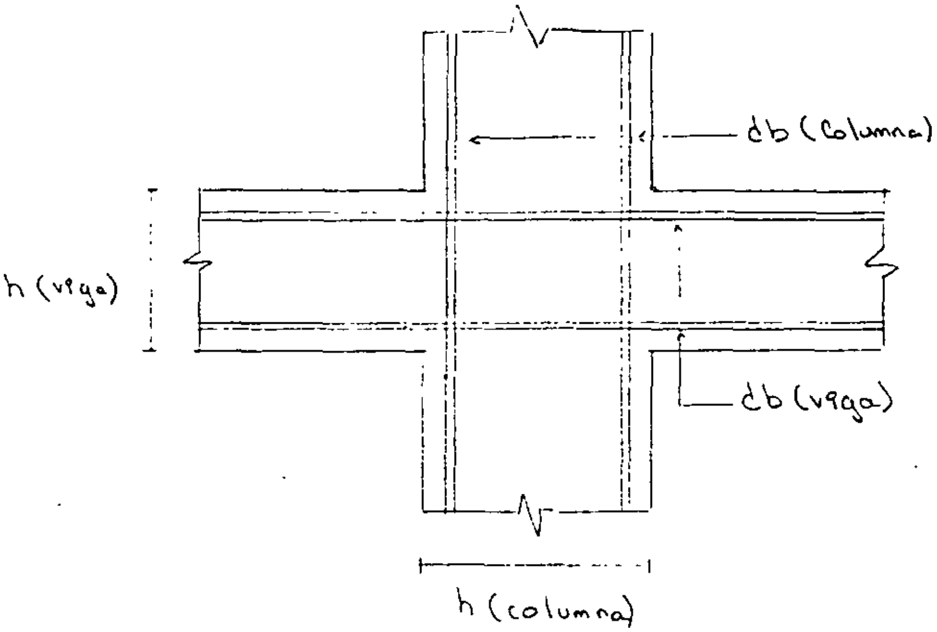
h(col.) es la dimensión transversal de la columna en la dirección de las barras de viga consideradas

Si en la columna superior del nudo cumple:

$$\frac{Pu}{Ag\,f'c} \geq 0.3$$

Entonces: $h(viga)/db(barras\ de\ col.) \geq 15$

NOTA: tambien ésta relación se cumplirá cuando en la estructura los muros de concreto reforzado resistan más del 50% de la fuerza lateral total.

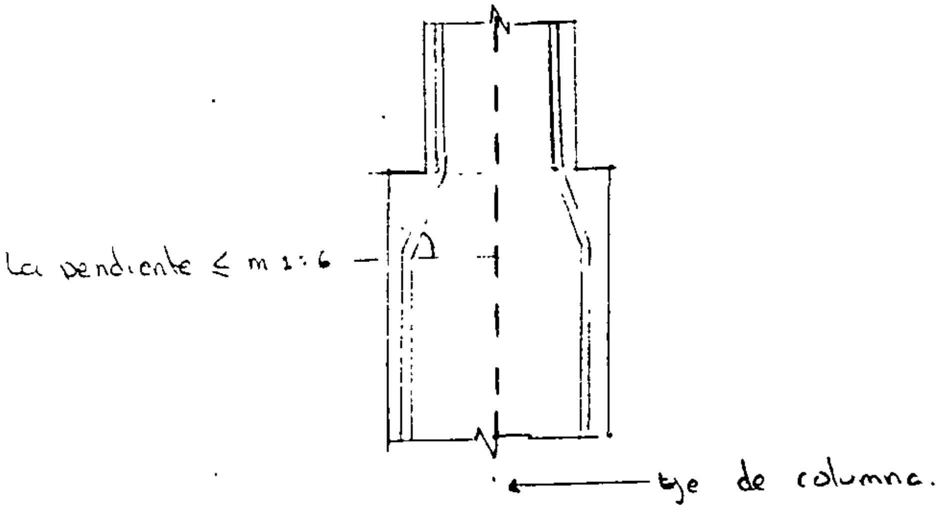


GENERALES:

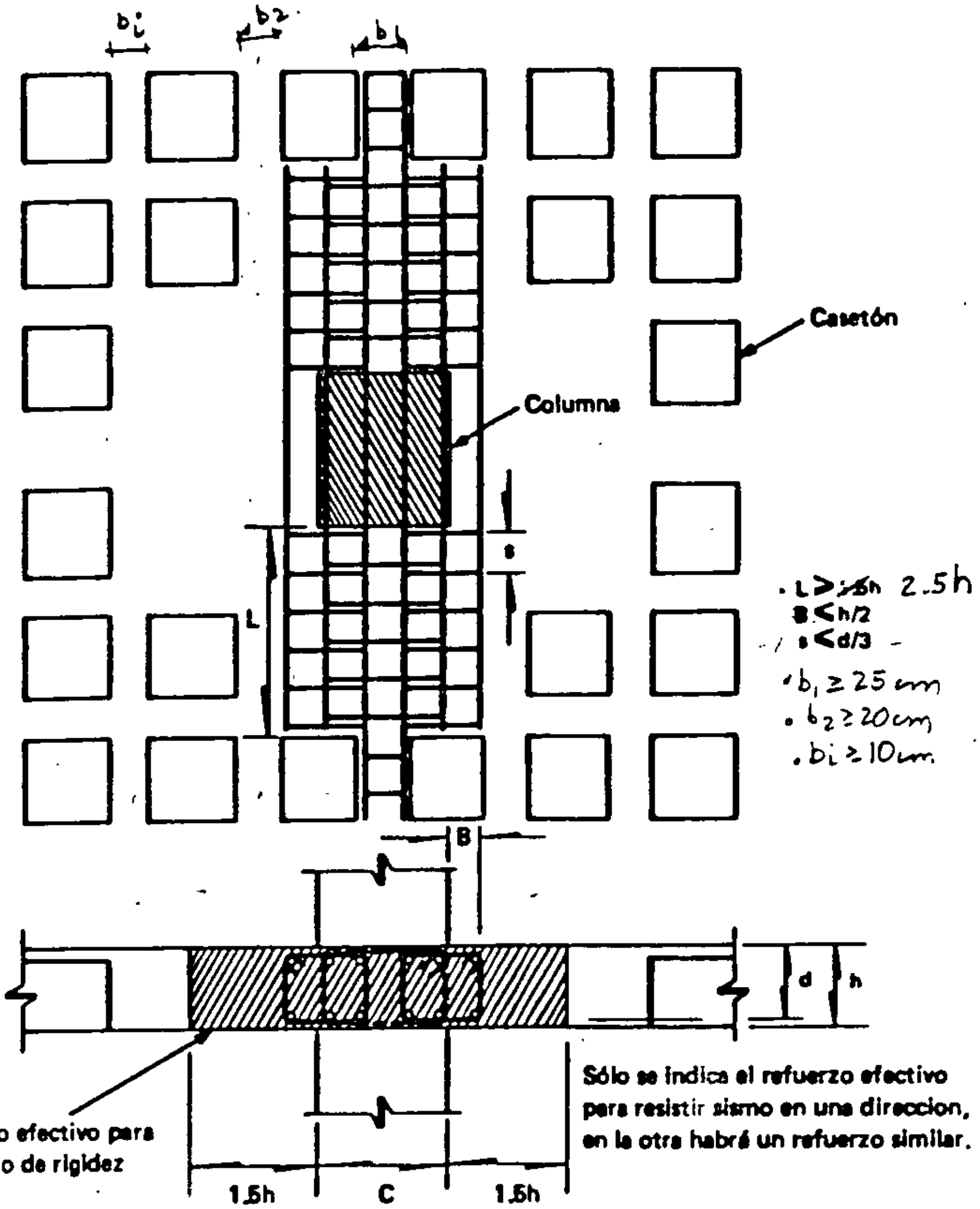
Será de la misma manera que en la zona de confinamiento (el diámetro del ref. transversal no será menor que los usados en la columna en las secciones próximas a dicha sección, si igual que en la zona de confinamiento).

Si la intersección es excéntrica, deben tomarse en cuenta las fuerzas cortantes, momentos y torsiones causados por la excentricidad.

10.16 c)



10.16 d)



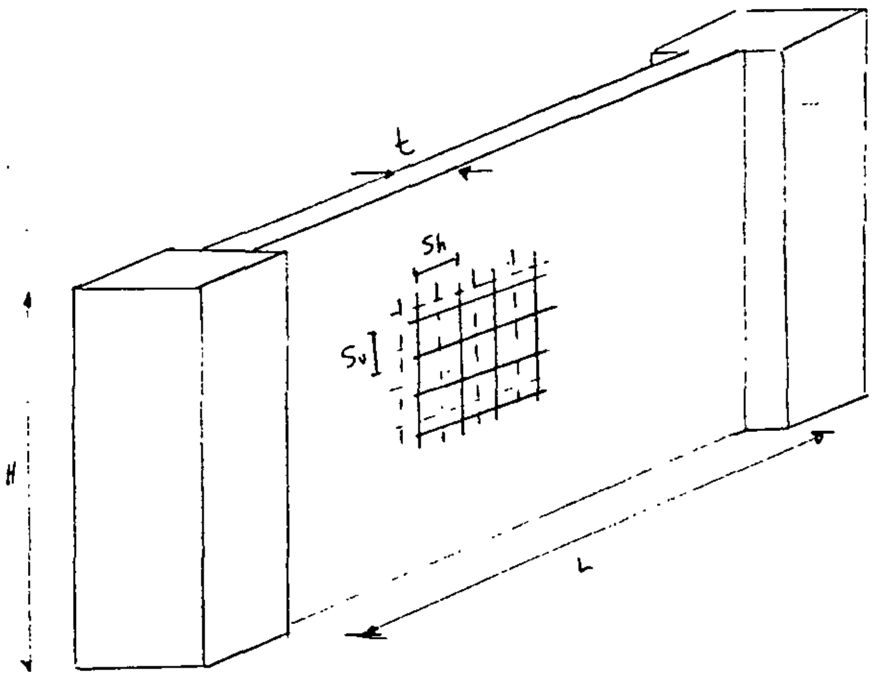
- $L \geq$ ~~5h~~ 2.5h
- $B \leq h/2$
- $s \leq d/3$
- $b_1 \geq 25$ cm
- $b_2 \geq 20$cm
- $b_i \geq 10$cm

Sólo se indica el refuerzo efectivo para resistir sismo en una direccion, en la otra habrá un refuerzo similar.

h (columna)/db (barras de losa) ≥ 20   h(columna) – es la dimensión transversal de la columna en la dirección de las barras de losa consideradas.

h (losa)/db (barras de col.) ≥ 15

Fig 10,17 Refuerzo en la conexión losa plana-columna



Limitaciones geométricas

$L/t \le 70$

$t \ge$ 13 cms

0.06 H (altura no restringida lateralmente).

Refuerzo mínimo

$p_v, p_h \ge 0.0025$

$S_v, S_h \le 35$ cm.

$p_h = \frac{A_{sh}}{t S_v}$ $p_v = \frac{A_{sv}}{t S_h}$

Fig. 10.18 Requisitos geométricos y de refuerzo en muros de concreto

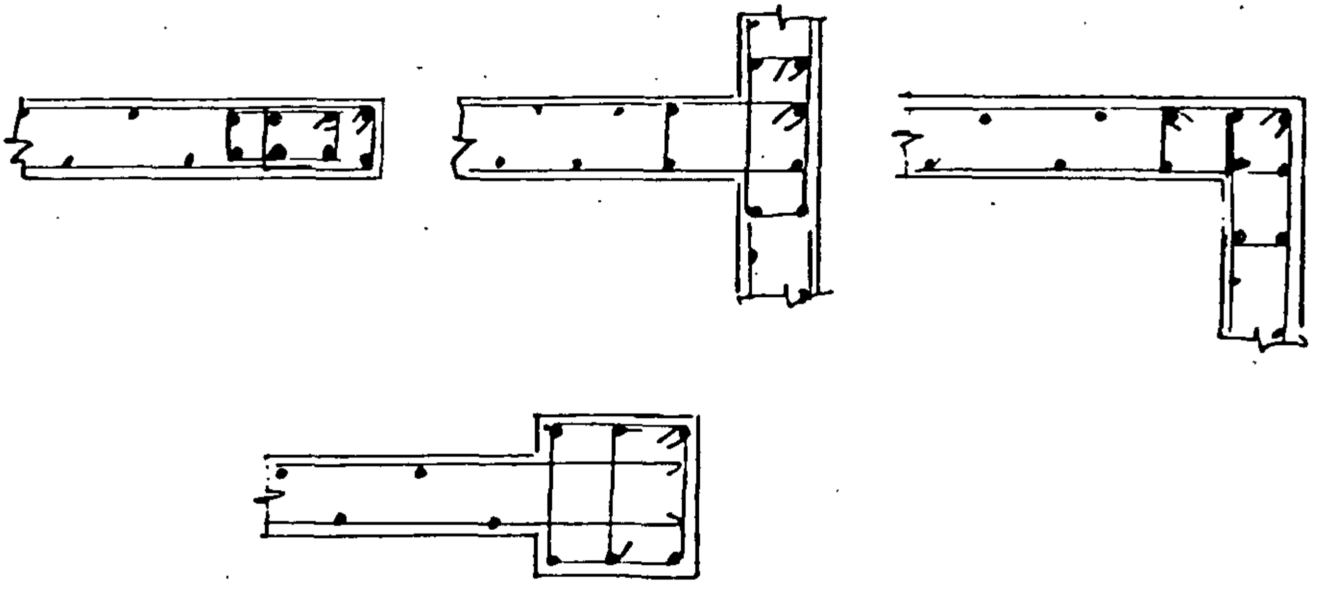
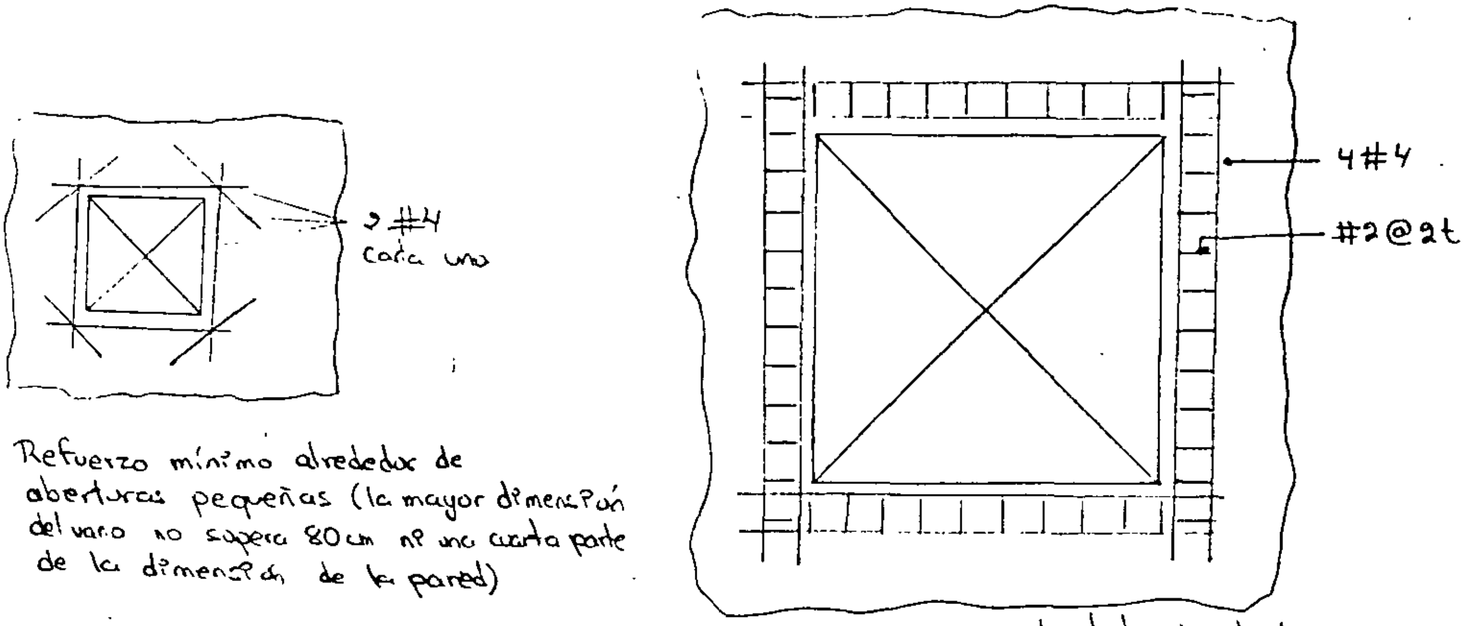


Fig. 10,19 Refuerzo en los extremos de muros



Refuerzo mínimo alrededor de aberturas pequeñas (la mayor dimensión del vano no supera 80 cm ni una cuarta parte de la dimensión de la pared)

Refuerzo mínimo alrededor de aberturas mayores.

Fig. 10.20 Detalles recomendados de refuerzo alrededor de aberturas en muros de concreto.

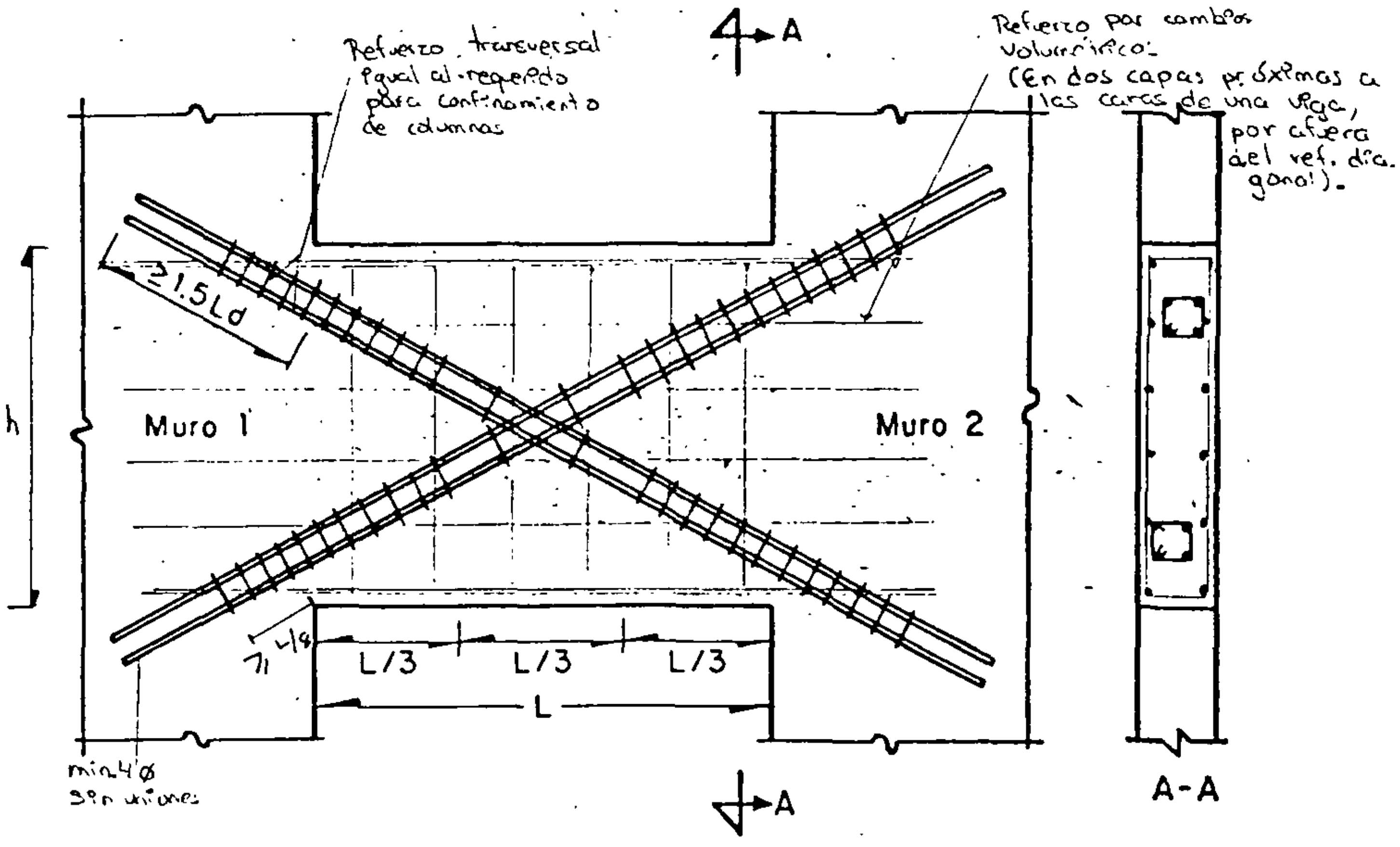


Fig. 10.21 Refuerzo de viga de acoplamiento que une muros de

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XXVI CURSO INTERNACIONAL DE INGENIERIA SÍSMICA

MOUDLO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

DISEÑO SISMICO DE ESTRUCTURAS DE CONCRETO

EXPOSITOR: DR. OSCAR LOPEZ BATIZ
PALACIO DE MINERIA
SEPTIEMBRE DEL 2000

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DISEÑO SISMICO DE EDIFICIOS

TEMA: Comportamiento Sísmico de Estructuras de Concreto

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Referencia de las notas presentadas para el tema.

1. Oscar López Bátiz; "Estructuras de Concreto Reforzado (I), Aspectos fundamentales sobre elementos viga y columna, comentarios sobre estructuras de cimentación"; Curso: Seguridad Sísmica de las construcciones para directores responsables de obra, CENAPRED, Sep. 1994.

2. Sergio M. Alcocer; "Comportamiento y Diseño de Estructuras de Concreto Reforzado, Muros Estructurales"; Curso: Seguridad Sísmica de las construcciones para directores responsables de obra, CENAPRED, Sep. 1994.

3. Sergio M. Alcocer; "Comportamiento y Diseño de Estructuras de Concreto Reforzado, Uniones de Elementos"; Curso: Seguridad Sísmica de las construcciones para directores responsables de obra, CENAPRED, Sep. 1994.

4. Vitelmo V. Bertero; "Seismic Behavior of Structural Concrete Linear Elements (Beams, Columns), and Their Connections"; Comité Euro-International du Béton, Bulletin d'Information No. 131, AICAP-CEB Symposium, Rome, Mai 1979, Vol.1 State of the Art Reports.

5. Oscar López Bátiz; "Avances en el Diseño Sísmico de Estructuras de Concreto Reforzado, Procedimiento de Diseño por Resistencia Ultima en Estructuras de Concreto Reforzado".

ESTRUCTURAS DE CONCRETO REFORZADO (I)
Aspectos fundamentales sobre elementos viga y columna, comentarios sobre estructuras de cimentación

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1. Introducción.

Las estructuras de concreto reforzado son estructuras heterogéneas producto de la combinación o mezcla de materiales con características diferentes como son el concreto y el acero de refuerzo, los procedimientos de diseño para estas estructuras consideran las propiedades de estos materiales en sus planteamientos. El uso del concreto reforzado comenzó en la segunda mitad del siglo XIX, desde los inicios de su empleo hasta la actualidad, la calidad de ambos materiales se ha incrementado, también las tecnologías de construcción y los procedimientos de diseño han avanzado de manera que las estructuras de concreto reforzado son de reconocida importancia en los ámbitos arquitectónico e ingenieril. Principalmente, en las décadas recientes los avances en el conocimiento sobre el comportamiento de los materiales y las estructuras de concreto reforzado han sido importantes.

Entre las propiedades importantes del concreto reforzado se encuentran la gran resistencia al fuego y efectos de intemperismo, la estabilidad de su durabilidad, el poco costo que requiere la supervisión durante su construcción, la versatilidad para su empleo en formas arquitectónicas caprichosas, propiedades que constituyen la fuerza que genera avances en la tecnología y conocimientos sobre el concreto reforzado. Así, a partir del inicio del siglo XX, prácticamente cada país cuenta con códigos y manuales propios para diseño y construcción de este tipo de estructuras.

Para lograr los códigos y manuales actuales para estructuras de concreto reforzado, en su elaboración y modificación se han incorporado tanto los materiales y técnicas comúnmente empleados, como aquellos avances logrados sobre calidad de los materiales y sobre las tecnologías de diseño y construcción. Sin embargo, debido a la velocidad con que se logran materiales nuevos y de mejor calidad, a la rapidez con que aparecen nuevas tecnologías en procesos constructivos y métodos de análisis estructural, surge la necesidad de revisar con mayor frecuencia la normatividad de diseño y construcción para este tipo de estructuras.

Respecto a procedimientos o métodos para diseño estructural de estructuras de concreto reforzado, a partir de 1953 el Comité Europeo del Concreto (CEB) inició un proceso de revisión de sus códigos y manuales. Durante los años 1964 y 1970, este comité introdujo un procedimiento nuevo, consistente en el diseño racional basado en la teoría de probabilidades y confiabilidad estructural.

Posterior a una recopilación de información experimental y teórica, se le dio forma a la ultima versión del código en 1991. Sin embargo, la muestra de información experimental todavía no es suficiente, provocando que el código CEB-1991 se constituya como una filosofía de diseño a considerar o una opción para desarrollar un diseño estructural lógico y razonado.

Los objetivos del diseño estructural, son proporcionar al dueño un inmueble que cubra las necesidades y con las características que desea, generalmente esas necesidades pueden resumirse como:

1) Asegurar, con una estructura, el proporcionamiento de un espacio vital para un propósito determinado.

2) Durante el período de vida útil, dicha estructura deberá satisfacer las condiciones de servicio para las que fue creada.

3) Los costos de construcción y mantenimiento, entre otros que conforman el costo total del inmueble, deberán tender a la optimización.

Respecto al segundo punto, las condiciones y características que deberá cubrir la estructura a diseñar variarán dependiendo de la función que se asigne al inmueble. En condiciones de servicio, una estructura deberá mantener su estabilidad ante grandes deformaciones y vibraciones. En el caso de estructuras de concreto reforzado, donde la presencia de grietas de gran apertura facilitaría el efecto del intemperismo en el acero de refuerzo y la consecuente degradación de resistencia y rigidez, es necesario limitar el ancho máximo de grieta dentro de un valor determinado, el cual es considerado al plantear las formulaciones de resistencia en los códigos y reglamentos. De igual manera, las condiciones de servicio de una estructura deberán mantenerse ante la incidencia de carga cíclica, como son los casos del sismo y el viento.

Ante carga sísmica o de viento, el diseño de las estructuras se plantea para que mantenga su estabilidad total ante la incidencia de la carga o efecto máximo esperado durante el período de vida útil del inmueble por efecto de dichos fenómenos naturales.

2. Materiales.

2.1 Concreto.

Dentro de las Características mecánicas que posee el concreto, la de mayor importancia es la resistencia a la compresión axial (σ_B). Las resistencias a tensión, flexión, cortante, de adherencia, así como el módulo de elasticidad del concreto, presentan una fuerte relación con la resistencia a la compresión axial, por lo que se considera a esta propiedad como la representativa del concreto.

Cuando al concreto se le sujeta a esfuerzos monotónicamente crecientes, la estructura del mismo va sufriendo de fracturamiento en su estructura interna, por lo que la curva esfuerzo-deformación de este material se presenta como la mostrada en la Fig.1, en esta figura la curva presenta una pendiente que decrece a mayor esfuerzo y, aproximadamente a 0.2% de deformación se alcanza la resistencia máxima a compresión del material, posterior a esta deformación los esfuerzos en el concreto decrecen con rapidez alcanzándose el aplastamiento a una deformación unitaria de 0.3 a 0.4 %.

Debido a que la curva esfuerzo-deformación del concreto no es lineal, para determinar el módulo de elasticidad del material existen diferentes procedimientos. Un procedimiento comúnmente empleado es definir el módulo de elasticidad del concreto a partir de la curva esfuerzo-deformación, definiéndose como la pendiente de la secante al origen del punto de la curva para un esfuerzo de 1/3 la resistencia a compresión. También, el módulo de elasticidad del concreto se define en la mayoría de los códigos y reglamentos de diseño como función de la resistencia a compresión registrada en un ensaye uniaxial, y sin considerar el efecto de creep en el material se definen las siguientes ecuaciones:

RDF $Ec = 14000 \times (\sigma_B)^{0.5}$ (kgf/cm^2)

ACI $Ec = \gamma^{1.5} \times 0.14\ (\sigma_B)^{0.5}$ (kgf/cm^2, para valores γ = 1440 a 2480 kgf/m^3)

AIJ $Ec = 2.1 \times 10^5 \times (\gamma/2.3)^{1.5} \times (\sigma_B/200)^{0.5}$ (kgf/cm^2)

donde: RDF: Reglamento del Distrito Federal, México; ACI: Reglamento del Instituto del Concreto de los Estados Unidos de Norteamérica; AIJ: Reglamento del Instituto de Arquitectos de Japón; Ec: módulo de elasticidad del concreto; γ: peso volumétrico del concreto.

La resistencia a tensión del concreto tiene estrecha relación con la resistencia a compresión, para resistencias a la compresión entre 180 y 240 kgf/cm^2 la resistencia a tensión generalmente se considera como 1/10 de la resistencia a compresión obtenida de pruebas uniaxiales en cilindros estándar.

La resistencia a compresión del concreto y su capacidad de deformación varían notoriamente de acuerdo a los esfuerzos confinantes a los que esté sujeto el material. Así, en la Fig.2, se presentan los resultados de pruebas a compresión realizadas en cilindros de concreto sujetos a esfuerzos confinantes proporcionados con líquido dentro de una cámara triaxial. Se observa un incremento notable de la resistencia a compresión de los cilindros, sin embargo, al considerar el concreto dentro de un elementos estructural donde el confinamiento se lo proporcione el acero de refuerzo longitudinal y principalmente transversal, la distribución de esfuerzos confinantes no presenta la misma mecánicos de la misma sección, producto de las cargas a las que se sujete el elemento (momento flexionante y carga axial).

El principio de Bernoulli es una hipótesis razonable en la zona a compresión del concreto, pero no es estrictamente aplicable en la vecindad del agrietamiento. Sin embargo, es aplicable a la deformación por tensión media de la zona agrietada. El principio de Bernoulli no se cumple totalmente en regiones sujetas a altos esfuerzos de cortante

3.1.2 Cálculo de la resistencia por flexión.

La resistencia ultima de un elemento bajo un estado de esfuerzos producto de flexión se define cuando la deformación unitaria máxima en el concreto de la fibra extrema a compresión de la sección transversal analizada alcanza un valor especificado, para el cual los reglamentos de diseño generalmente recomiendan valores que varían de 0.003 a 0.004. La mayoría de los reglamentos de construcción hipotetizan la distribución de esfuerzos a compresión en la sección transversal como rectangular definida por dos o tres parámetros, de tal modo que la resultante de esta distribución rectangular y la resultante y la posición de la misma, producto de considerar la curva esfuerzo - deformación "real" uniaxial del concreto, sea la misma. Así, se presentan distribuciones esfuerzo - deformación simplificadas en secciones sujetas a flexión como las presentadas en la Fig.7, donde se muestran aquellas distribuciones adoptadas en las reglamentos RDF, ACI y AIJ. Resultando en fórmulas para el cálculo de resistencia última como las siguientes:

RDF $M = (A_{st} - A_{sc})\ f_y\ (d - a/2) + A_{sc} \times f_y\ (d - d')$

si, $(p_t - p_c) \geq [\ 4800\ /\ (6000 - f_y)\ ] \times [\ (d' \times 0.85\ \sigma_B)\ /\ (d \times f_y)\ ]$

Cuando no se cumpla, M se determinará con un análisis de la sección basado en las hipótesis básicas de análisis por flexión.

ACI si, $(A_{st} - A_{sc})\ /\ (b \times d) \geq 0.85\ \beta_1\ [\ (\sigma_B \times d')\ /\ (f_y \times d)\ ]\ 6115\ /\ (6115 - f_y)$

$M = (A_{st} - A_{sc})\ f_y\ (d - a/2) + A_{sc} \times f_y\ (d - d')$

donde, $a = (A_{st} - A_{sc})\ f_y\ /\ (0.85\ \sigma_B \times b)$

si, $(A_{st} - A_{sc})\ /\ (b \times d)$ es menor que el valor indicado, A_{sc} puede no considerarse.

AIJ $M = \sigma_B \times b \times D^2\ \{g_1\ (A_{st} \times f_y)/(\sigma_B \times b \times D)\ \}$

donde, A_{sc}: área del acero de refuerzo en compresión; b: ancho de la sección transversal del elemento; d: peralte efectivo de la sección transversal; p_t: cantidad de acero de refuerzo longitudinal a tensión; p_c: cantidad de acero de refuerzo longitudinal a compresión; β_1: factor que depende de la resistencia a compresión del concreto ($\beta_1 = 0.85$ para $\sigma_B \leq 280$ kgf/cm^2, el valor de β_1 disminuye en 0.05/70 kgf/cm^2); d': dimensión del recubrimiento de concreto; g_1: distancia entre los centroides de las barras a tensión y compresión/D; D: peralte de la sección; A_{st}: área del acero de refuerzo a tensión; f_y: esfuerzo de fluencia del acero a tensión.

3.1.3 Factores que afectan la resistencia, capacidad de deformación y ductilidad de un elemento estructural a flexión.

Como se puede discernir de las fórmulas para calcular la resistencia de elementos de concreto reforzado sujetos a flexión, la resistencia a la fluencia por tensión del acero de refuerzo empleado y la resistencia a la compresión del concreto , junto con las dimensiones de la sección transversal, determinan básicamente la resistencia de un elemento de concreto reforzado ante el agrietamiento, la fluencia y la carga última por flexión.

Como se aprecia en la Fig.8, donde se muestra una representación típica de una relación momento-curvatura para una sección transversal de un elemento de concreto con una cantidad de acero de refuerzo menor o igual al acero de refuerzo para la falla balanceada (falla balanceada es aquella cuando el concreto alcanza su deformación unitaria máxima al mismo tiempo que el acero de refuerzo fluye por tensión), otros parámetros importantes que determinan

la calidad estructural del elemento son la capacidad de deformación (considerada como la ductilidad del elemento) y su capacidad para disipar energía incidente ante carga monotónicamente creciente o cíclica.
La ductilidad o capacidad de deformación post-fluencia, se define como la relación entre la deformación (curvatura) última alcanzada y la deformación (curvatura) al punto de fluencia:

$$\mu = \phi_u / \phi_y$$

Asumiendo una relación esfuerzo-deformación, se determina la posición del eje neutro al alcanzar el momento de fluencia en la sección (c_1). Por relaciones trigonométricas se define la curvatura a la fluencia como se indica:

$$\phi_y = \varepsilon_y / d \ (1 - c_1)$$

donde, ε_y: deformación unitaria a la fluencia por tensión en el acero de refuerzo.

Siguiendo un procedimiento similar se determina la posición del eje neutro al alcanzar la resistencia última de flexión (c_{1u}), y se determina la curvatura ultima de la sección transversal:

$$\phi_u = \varepsilon_{cu} / (c_{1u} \times d)$$

donde: ε_{cu}: es la deformación unitaria en la fibra extrema del concreto a compresión en la sección transversal.

Para secciones transversales rectangulares se pueden definir las expresiones para el calculo de la posición del eje neutro medido desde la fibra a compresión, c_1 y c_{1u}, como se muestra:

$$c_1 = [\ (n \times p_t)^2 + 2 \times n \times p_t\ ]^{0.5} - n \times p_t$$

$$c_{1u} = (p_t - p_c)\ f_y / \sigma_B$$

donde, $n = E_s/E_c$, E_s es el módulo de elasticidad del acero de refuerzo y E_c es el módulo de elasticidad del concreto; p_t: cantidad de acero de refuerzo a tensión ($= A_{st} / b \times d$); p_c: cantidad de acero de refuerzo a compresión ($= A_{sc} / b \times d$).

Analizando la última expresión, se aprecia que la presencia del acero de refuerzo a compresión, contribuye a disminuir la localización del eje neutro respecto a la fibra extrema a compresión, provocando un aumento en la curvatura última y por tanto mayor capacidad de deformación. La ductilidad de la sección transversal se puede definir entonces:

$$\mu = \phi_u / \phi_y = \varepsilon_{cu}\ (1 - c_1) / \varepsilon_y\ (c_{1u})$$

Definiendo al parámetro "índice de refuerzo q (= $[p_t - p_c]\ f_y / \sigma_B$)" y, por medio de las expresiones para determinar c_1, c_{1u} y μ, se establece una relación entre este índice de refuerzo q y la ductilidad de la sección transversal analizada μ, se obtienen gráficas como las mostradas en la Fig.9. De la gráfica se entiende lo siguiente: a) a menor valor del índice de refuerzo, la capacidad de ductilidad del elemento aumenta; b) a mayor cantidad de acero de refuerzo en la zona a compresión, mayor será la capacidad de deformación post-fluencia del elemento; c) conjugando adecuadamente los dos aspectos anteriores, se puede asegurar que un elemento presente falla por flexión con suficiente capacidad de deformación post-fluencia (ductilidad).
La capacidad de absorber energía por medio de deformación se puede cuantificar, de una relación momento-curvatura monotónicamente creciente como la de la Fig.8, como el área comprendida bajo dicha curva.
Considerando un segmento de longitud unitaria del elemento estructural, cuyo comportamiento se asume elasto-plástico perfecto, la energía absorbida por deformación se puede representar con la siguiente expresión:

$$U = \sigma_B \times b \times d\ [\varepsilon_{cu} - c_{1u} \times \varepsilon_y / 2\ (1 - c_{1u})\ ]\ (1 - 0.425\ c_{1u})$$

Analizando la expresión anterior se puede concluir que el incremento de las dimensiones de la sección transversal y la resistencia a la compresión del concreto, afectan en proporción directa el incremento en la capacidad de absorción de energía del elemento. Igualmente, incrementado la deformación unitaria última del concreto en la sección transversal, lo que se puede lograr con mayor y mejor colocación del acero de refuerzo lateral, se obtiene el efecto de incrementar la capacidad de deformación post-fluencia y, por lo tanto, la capacidad de absorción de energía es mayor. Comparativamente con los parámetros anteriores, incrementos en p_t y f_y no presentan un efecto directo en la capacidad de deformación y de absorción de energía de los elementos estructurales. De la expresión para el cálculo de c_{1u} se concluye que aumentando la cantidad de acero a compresión el valor de c_{1u} disminuye, lo que repercute en incrementar la capacidad de absorber energía del elemento.

Cabe mencionar que los factores antes citados tienen un efecto similar en elementos sujetos a efectos de flexo-compresión. En el caso de estos últimos, resulta también de gran importancia el confinamiento adecuado del concreto del núcleo y el proporcionar límites permisibles de carga axial.

3.2 Comportamiento de elementos sujetos a flexocompresión.

En estructuras a base de marco momento resistentes, los elementos columna en la mayoría de los casos estarán sujetos a carga axial y momento flexionante (uniaxial y biaxial). En otros casos, aunque teóricamente la columna este sujeta únicamente a carga axial, por problemas de control de calidad en la etapa constructiva se generan desviaciones en el dimensionamiento y distribución de las secciones transversales, provocando excentricidad de la carga axial respecto al eje del elemento, lo que genera momento flexionante a considerar en el diseño de dicho elemento.

Respecto a las hipótesis básicas para análisis de elementos sujetos a flexo-compresión, estas son exactamente las mismas que aquellas consideradas para elementos bajo flexión simple.

3.2.1 Diagrama de interacción.

La resistencia de la sección transversal a una fuerza de compresión, se reduce con la presencia del momento flexionante. El diagrama de interacción representa el lugar geométrico de los puntos que indican la carga axial y momento flexionante que provocan que un elemento alcance su resistencia última (deformación unitaria última en la fibra extrema a compresión de la sección transversal), su representación gráfica se muestra en la Fig.10. Así, para una curva de interacción determinada, si la columna esta sujeta a una combinación de momento flexionante y carga axial que esté en el interior de dicha curva, el elemento se encuentra del lado de la seguridad. Contrariamente, si la combinación está fuera de la curva, la columna estará propensa a la falla.

Como se muestra en la Fig.10, existen tres puntos importantes que definen las características de resistencia en el diagrama de interacción de una columna. El punto localizado donde el diagrama intersecta al eje vertical, corresponde a una columna sujeta únicamente a compresión axial, pero producto de la excentricidad existente por problemas intrinsecos al proceso de construcción, se recomienda para diseño el uso de un valor mínimo de excentricidad de diseño (e_{min}), dando como resultado una disminución de la resistencia por compresión hasta alcanzar la curva original en el punto A. El punto B representa el estado de falla denominado "falla balanceada", en el cual la deformación unitaria última en el concreto a compresión se alcanza al mismo tiempo que el acero de refuerzo fluye a tensión. Al momento flexionante y carga axial representativos de este punto se les llama momento y carga balanceada, y a la relación entre el momento flexionante y la carga axial correspondiente a esta falla balanceada se le denomina excentricidad balanceada (e_b). El diagrama de interacción se intersecta con el eje horizontal en el punto que representa al elemento en flexión pura, representando obviamente el momento de falla por flexión uniaxial. La parte del diagrama de interacción correspondiente al elemento bajo tensión axial y flexión, se calcula de la misma manera que para compresión y flexión, sin embargo, es un estado poco común e indeseable en el diseño de estructuras de concreto de mediana altura.

La determinación del punto de la falla balanceada es importante desde el punto de vista de los reglamentos para diseño estructural, porque para elementos cuya excentricidad sea menor que la excentricidad balanceada (e_b), antes que el acero de refuerzo longitudinal fluya por tensión se presenta la falla por aplastamiento en el concreto sujeto a esfuerzos de compresión, denominándose a este rango de "falla por aplastamiento" (es una falla de tipo poco dúctil). Si la excentricidad en el elemento es mayor que la balanceada, se encuentra en el rango de "falla por tensión" (falla con características dúctiles).

En la Fig.11 se muestra, para un diagrama de interacción carga axial - momento flexionante de una sección transversal determinada, el correspondiente diagrama de curvatura última calculada. Se aprecia que para cargas axiales menores que la carga axial correspondiente a la falla balanceada, la curvatura correspondiente al momento de falla presenta un brusco incremento. Contrariamente, si la carga axial incidente es mayor que la balanceada, la capacidad de deformación post-fluencia tiende a ser nula. Es por lo anterior que al diseñar una columna ante efectos sísmicos se proponga un límite en la carga axial permisible.

En el caso de columnas sujetas a flexión biaxial y compresión existen dos formas típicas de encontrar la resistencia última de estos elementos. Una es la solución por tanteos, que consiste en encontrar el valor máximo de la carga axial "P" que actúa fuera de dos planos de simetría, a excentricidades e_x y e_y. Esta condición es equivalente a considerar una carga axial P y dos momentos flexionantes, $M_x = P \times e_x$ y $M_y = P \times e_y$. Para un elemento con geometría y excentricidades dadas, aplicando el procedimiento básico para elementos sujetos a flexo-compresión partiendo de conocer las características esfuerzo-deformación de los materiales, iterativamente se puede obtener el valor máximo de la carga P que actúa a las excentricidades dadas. Este proceso predice satisfactoriamente la resistencia del elemento, pero es muy laborioso. Sin embargo, para casos particulares comúnmente empleados en la práctica, se han desarrollado diagramas de interacción empleando computadora electrónica y se muestran en algunos reglamentos. La otra forma de obtener la resistencia última de este tipo de elementos es aproximada, siendo un ejemplo típico la llamada "Fórmula de Bresler". Bresler desarrolló una expresión simple para calcular los valores máximos de carga axial de compresión que actúa a excentricidades e_x y e_y en secciones rectangulares con distribución simétrica de refuerzo longitudinal. La expresión que propone es la siguiente:

$$1/P_r = 1/P_x + 1/P_y - 1/P_0$$

donde, P_r: carga normal máxima que actúa a excentricidades e_x y e_y; P_x: carga normal máxima a una excentricidad e_x ($e_y = 0$); P_y: carga normal máxima a una excentricidad e_y ($e_x = 0$); P_0: carga axial máxima que puede resistir el elemento ($e_x = e_y = 0$).

Es evidente que el problema se reduce a una combinación de soluciones mas simples, dos de flexo-compresión y una de compresión axial.

3.2.2 Cálculo de resistencia a flexo-compresión.

De la misma manera que para el caso de flexión, la resistencia última del elemento se determina cuando la deformación unitaria máxima en el concreto de la fibra extrema a compresión alcanza un valor especificado que varía entre 0.003 a 0.004 (según el reglamento empleado).

En el reglamento del AIJ, se plantea una fórmula simplificada para el cálculo de resistencia última de columnas a flexo-compresión, que se presenta enseguida:

$$\text{AIJ} \qquad M = \sigma_B \times b \times D^2 \{g_1 (A_{st} \times f_y) / (\sigma_B \times b \times D) + 0.5 \times [N / (\sigma_B \times b \times D)] \times [1 - N / (\sigma_B \times b \times D)]\}$$

expresión con confiabilidad de ± 20% en el 90% de su comparación con resultados experimentales. Expresión válida para :

$$p_t = A_{st} / (b \times D) \text{ ; entre } 0.4 \text{ y } 2.8\ \%$$

$$N / (b \times D) \leq N_b / (b \times D)$$

donde, N: carga axial actuando en el elemento; Nb: Carga axial en la condición de falla balanceada.

4. Comportamiento de elementos lineales (vigas y columnas) ante fuerza cortante.

La falla por cortante en elementos de concreto reforzado, a diferencia de la falla por flexión, es repentina y

generalmente produce un estado de inestabilidad irreparable en el elemento en particular y la estructura en general. Por lo que los procedimientos de diseño presentados en los reglamentos tienden a tratar de eliminar este tipo de falla y lograr un factor de seguridad lo mayor posible respecto a este comportamiento indeseable en la estructura.
Respecto al mecanismo que define la falla por cortante en elementos de concreto reforzado, al contrario de la falla por flexión, es de mayor complejidad, y aunque se ha realizado mucha investigacion sobre el tema permanecen muchos puntos sin tener plena explicación. Es por eso que los procedimientos de diseño por cortante indicados en la mayoría de los reglamentos tienen una fundamentación empírica, apoyada con conceptos teóricos substraídos de la teoría de la elasticidad de los materiales y recientemente de la teoría de la plasticidad aplicada al concreto reforzado.

4.1 Modos de falla por cortante.

4.1.1 Vigas con relación claro a peralte grande ($a/d > 2.5$, donde "a" es la longitud del claro de corte).

a) Falla por flexión (Fig.12.a). Las grietas por efectos de flexión se propagan convirtiéndose en agrietamiento por efectos flexo-cortantes, extendiéndose a través del elemento causando una falla brusca por tensión diagonal.
b) Falla por tensión diagonal (Fig.12.b). En este tipo de falla no se observa ninguna de las características antes citadas, no se presenta tampoco aplastamiento del concreto a compresión, es una falla frágil e inestable. Incrementando la cantidad de refuerzo lateral se reduce considerablemente la posibilidad ocurrencia de este tipo de falla y se logran ductilidades que varían desde 1 hasta 4.

4.1.2 Vigas con relación claro a peralte pequeño, también llamadas vigas cortas ($1 < a/d < 2.5$).

a) Falla de tensión por cortante (Fig.13.a). EL agrietamiento por problemas de adherencia entre el acero de refuerzo y el concreto se propaga a lo largo del refuerzo longitudinal empezando en el extremo de la grieta inclinada de cortante. Como mecanismos resistentes importantes ante este tipo de falla se pueden citar el efecto de dovela del refuerzo longitudinal, la adherencia acero-concreto y la resistencia a deslizamiento acero-concreto. Reduciendo la relación p_t / p_w (donde p_t es la cantidad de acero de refuerzo longitudinal a tensión y p_w es la cantidad de acero de refuerzo lateral), se tiende a eliminar este tipo de falla. Igualmente, para un porcentaje de acero longitudinal dado, empleando barras de menor diámetro y distribuidas adecuadamente (sin emplear paquetes de barras) se logran comportamientos adecuados, reduciendo la posibilidad de ocurrencia de este tipo de falla.
b) Falla de compresión por cortante (Fig.13.b). En este tipo de falla el concreto a compresión en las fibras extremas de la sección transversal, en los extremos de las grietas de cortante, sufre aplastamiento y falla. Este problema se recrudece cuando el elemento se sujeta a niveles altos de carga axial y cuando se trata de elementos cortos de gran peralte. Una forma de evitar o aliviar este tipo de fallas es proporcionando un mayor porcentaje de acero lateral que lo proporcione mayor confinamiento al concreto sujeto a esfuerzos de compresión.

4.1.3 Vigas de gran peralte ($a/d < 1$).

En esta clase de elementos se generan esfuerzos significativos de compresión en los estratos de concreto resultantes entre las grietas inclinadas provocadas por efecto de cortante, y grandes esfuerzos de tensión a través de dichas grietas. Este fenómeno puede provocar:
a) Fallas de anclaje del acero a tensión, combinada con desprendimiento del concreto de recubrimiento por efecto de dovela.
b) Falla por aplastamiento del concreto en los apoyos.
c) Falla de flexión debido a la rotura post-fluencia del acero de refuerzo longitudinal, o al aplastamiento del concreto en la parte superior del mecanismo de arco.
d) Falla por aplastamiento en el concreto de los estratos a compresión ubicados en la vecindad del agrietamiento diagonal por cortante.

4.2 Principales mecanismos de resistencia al cortante.

4.2.1 Equilibrio en el claro de cortante de la viga (sin refuerzo lateral).

La resistencia a cortante estará determinada por los siguientes mecanismos de transferencia de cortante: a) Transmisión de fuerza cortante a través del concreto en la zona a compresión, V_c; b) Resistencia por efecto de dovela, transmitida través de la grieta por el refuerzo longitudinal, V_d; c) Resistencia al cortante producto de la componente vertical los esfuerzos cortantes inclinados υ_a, transmitidos a través de las grietas inclinadas por medio de cortante directo o cizalleo entre las partículas de agregados que se encuentran entre y/o en las superficies de la grieta, V_a.

$$V = V_c + V_a + V_d$$

$$M = x\ V = j_d\ (T + V_d \cot\alpha)$$

donde, el significado de todas las variables y parámetros se muestran en la Fig.14.

De trabajos experimentales se ha concluido que el efecto de dovela es pequeño para elementos sin acero de refuerzo lateral; $V_d = 0$, por lo que la expresión anterior se puede representar como sigue

$$M = T \times j_d$$

La fuerza de tensión en el refuerzo longitudinal a una distancia ($x - j_d \cot\alpha$) es determinada por el momento a una distancia x desde el apoyo del elemento estructural. El incremento de esfuerzos en el acero, claramente depende de la pendiente de la curva diagonal con la que se idealizó al agrietamiento por efecto flexo-cortante,

$$V = dM / dx = d(T\ j_d) / dx = j_d\ dT / dx + T\ d(j_d) / dx$$

donde, $j_d\ dT / dx$: representa el comportamiento de un elemento prismático sujeto a flexión, en el cual la fuerza de tensión interna T actúa sobre un brazo de palanca constante j_d. A este efecto se le denomina efecto de viga dentro de los mecanismos de transmisión de fuerza cortante; $T\ d(j_d) / dx$: representa el comportamiento de un arco tensado, en el que la fuerza cortante externa es resistida por el estrato interno de concreto a compresión. A este efecto se le conoce como efecto de arco; dT/dx: variación de la fuerza interna de tensión (fuerza de adherencia)

4.2.2 Acción de viga en el claro de cortante.

$$V_B = j_d\ dT/dx$$

En la acción de viga intervienen los siguientes tipo de mecanismos de transferencia de fuerza cortante:

a) Fuerza de adherencia, $\Delta T = T_1 - T_2$

b) Cortante transmitido por cortante directo o cizalleo en el agregado localizado en las caras de la grieta, υ_{a1}, υ_{a2}

c) Fuerzas por efecto de dovela, V_{d1}, V_{d2}

El momento en el cantiléver provocadas por las fuerzas de adherencia ΔT, son resistidas por el efecto de dovela y las fuerzas generadas en el agregado, a las que se adiciona la resistencia a flexión M_c del concreto mismo en la zona de compresión. La representación de estos fenómenos claramente se aprecia en la Fig.15.

En vigas con dimensiones normales, un máximo de 20% de las fuerzas por adherencia pueden ser resistidas por flexión en el concreto. Sin acero de refuerzo en el alma, la capacidad del efecto de dovela se limita a la resistencia a tensión del concreto siendo prácticamente nula. Cuando se cuenta con acero de refuerzo lateral, la contribución del efecto de dovela no excede el 25% de la resistencia total del cantiléver. El ancho de la grieta, la rugosidad de su superficie, la deformación por cortante y la resistencia del concreto determinan la resistencia por cortante directo o cizalleo en el agregado, siendo aproximadamente del 50 al 70% de las fuerzas por adherencia las que se resisten por este efecto del agregado en la superficie de la grieta.

El cortante transmitido por la zona de concreto a compresión, arriba de la grieta diagonal, incrementa lentamente durante los procesos de carga hasta alcanzar un máximo de 25 a 40% del total de la fuerza cortante incidente en la sección transversal de la viga.

4.2.3 Acción de arco en el claro de cortante.

$$V_a = T\ d(j_d)/dx$$

En este mecanismo la fuerza cortante se transmite por compresión diagonal en el concreto de los elementos estructurales. La intensidad de los esfuerzos de compresión diagonal dependen de la inclinación del campo de esfuerzos principales. La relación entre claro de cortante y peralte de la sección (a/d) define una medida de esta inclinación (Fig.16). El efecto del mecanismo de arco resulta inefectivo cuando la fuerza de cortante se transmite a la zona a tensión.

El tipo de fallas que se presentan en un elemento estructural por el mecanismo de arco son:

a) La propagación de las grietas inclinadas de cortante reducen la zona a compresión excesivamente, provocando aplastamiento del concreto sujeto a esfuerzos de compresión.

b) La línea de esfuerzos a compresión puede presentar excentricidad respecto al eje del elemento, y generar una falla de tensión por efecto de flexión en la "zona a compresión".

c) Cuando la relación entre claro de cortante y peralte de la sección es pequeña ($a/d < 2$), se observa una considerable reserva de resistencia producto de mayor eficiencia en el mecanismo de arco. En estos casos se presentará una falla por aplastamiento del concreto en el estrato diagonal a compresión, o falla por problema de adherencia y anclaje en el refuerzo longitudinal.

4.2.4 Papel del acero de refuerzo lateral en el comportamiento de un elemento estructural.

Los estribos o acero de refuerzo lâteral contribuyen en los mecanismos de resistencia ante fuerza cortante de la siguiente manera:

a) Contribuye a incrementar la resistencia por el efecto de dovela y disminuir la deformación relativa entre las caras de la grieta.

b) Disminuye los esfuerzos de tensión por flexión en los voladizos formados entre los agrietamientos del elemento (ver Fig.15), mediante una fuerza diagonal de compresión producto del efecto de armadura.

c) Limita la apertura de las grietas diagonales al rango elástico, favoreciendo la transferencia de cortante por cortante directo o cizalleo en el agregado localizado en la superficie de la grieta.

d) Proporciona confinamiento al concreto de la zona a compresión, incrementando su resistencia a compresión.

e) Previene un brusco decremento en la resistencia por adherencia cuando el agrietamiento por problema de adherencia y anclaje se desarrolla en zonas de anclaje y/o traslape.

4.3 Cálculo aproximado de la resistencia por cortante.

Para determinar la resistencia ante cortante han surgido una serie de investigaciones de gran importancia en el campo del concreto reforzado. Así, como punto inicial de la teoría de los mecanismos de transmisión y resistencia de fuerza cortante está el concepto de "analogía de la armadura" propuesto por Ritter en 1899, pasando por los trabajos de Ritter y Morch de 1903 sobre la transmisión del total de la fuerza cortante por el refuerzo lateral del elemento. También de importancia es el trabajo desarrollado por Talbot en 1909 donde plantea que 2/3 de la fuerza cortante incidente la resiste el refuerzo lateral y el 1/3 restante el concreto en la zona a compresión. En los años 50's, Walther y Morrow seguían haciendo trabajos analíticos importantes. Igualmente, Kani en el período comprendido entre 1964 y 1969 realizó trabajos teórico-experimentales encaminados a desglosar cualitativa y cuantitativamente los mecanismos que intervienen en la transmisión de la fuerza cortante incidente en un elemento de concreto reforzado, introduciendo el concepto de "acción de viga" (también conocido como modelo de diente) del segmento de concreto localizado entre dos grietas producto de efecto flexo-cortante. Todos estos estudios, básicamente no han cambiado el concepto primario propuesto por Ritter y Morch, y es en base a estos conceptos y resultados que la mayoría de los códigos y reglamentos actuales proponen sus formulaciones para el cálculo de la resistencia por cortante de elementos de concreto reforzado.

A continuación se presentan las fórmulas básicas propuestas en los reglamentos del DDF, ACI y AIJ:

RDF para $p_t \leq 0.01$

$$V_u = b \times d \,(0.2 + 30\, p_t)\, (\sigma_B)^{0.5} + A_w \times f_{wy} \times d\, (\text{sen}\theta + \cos\theta)$$

para $p_t \geq 0.01$

$$V_u = 0.5 \times b \times d\, (\sigma_B)^{0.5} + A_w \times f_{wy} \times d\, (\text{sen}\theta + \cos\theta)$$

ACI $V_u = V_c + V_t \leq 2.12\, (\sigma_B)^{0.5}\, b \times d$

$$V_t = p_w \times f_{wy} \times b \times d$$

$$V_c = 0.53\, [\, 1 + 0.0071\, N / (b \times d)\,]\, (\sigma_B)^{0.5}\, b \times d$$

IAJ $Vu = \{\, 0.068 p_{IAJt}{}^{0.23} (\sigma_B + 180) / (M/(Q \times d) + 0.12) + 2.7 (p_w f_{wy})^{0.5} + 0.1\, N / (b \times D)\, \}\, b \times j$

donde, A_w: área del acero de refuerzo lateral en una vuelta del mismo; f_{wy}: esfuerzo de fluencia del acero de refuerzo lateral; θ: ángulo de inclinación del refuerzo lateral respecto al eje del elemento; N: carga axial en el elemento; b: ancho de la sección transversal; p_{IAJt}: porcentaje de refuerzo por flexión, acero de refuerzo a tensión (%); M/Q: relación entre momento y fuerza cortante en la sección transversal; p_w: cantidad de acero de refuerzo lateral (= A_w / b x s); j: distancia entre las resultantes de esfuerzos a compresión y tensión en la sección transversal (puede considerarse como, j = 7d /8); s: espaciamiento del acero de refuerzo lateral; D: Peralte total de la sección transversal; unidades kgf, cm

Normalmente, la contribución de la losa a la resistencia ante fuerza cortante en vigas puede considerarse no significativa.

En todos los reglamentos se acepta que la carga axial, dentro de los límites permisibles por flexo-compresión, contribuye a incrementar la resistencia por cortante de los elementos columna. Sin embargo, en todos los códigos se hace una consideración puramente empírica sobre dicho efecto, lo que se aprecia claramente en las formulaciones de los reglamentos ACI y AIJ. Respecto al reglamento RDF se plantea el siguiente factor correctivo, que presenta gran similitud al considerado en el ACI:

$$P = 1 + 0.007\, (P_u / A_g)$$

para, $P_u \leq 0.7\, \sigma_B + 2000\, A_u$

si, $P_u \geq 0.7\, \sigma_B + 2000\, A_u$, el valor de P se variaría hasta ser nulo para

$P_u = A_g \times 0.85\, \sigma_B + A_s \times f_y$

donde, P_u: carga axial en el elemento; A_g: área de la sección transversal del elemento; A_s: área total del acero de refuerzo longitudinal en el elemento.

5. Propiedades de adherencia y anclaje acero - concreto.

Para que un elemento de concreto reforzado se considere monolítico, o trabaje como tal, es necesario la existencia de adherencia entre los materiales. Cuando el esfuerzo en el acero de refuerzo embebido en el concreto cambia, esa diferencia de esfuerzos deberá transferirse al concreto por medio de adherencia y anclaje. Los esfuerzos de adherencia y anclaje son esfuerzos de cortante desarrollados en la frontera entre la barra de acero y el concreto que la circunda para transmitir la fuerza, producto de la diferencia de esfuerzos antes mencionada, entre ambos materiales.

Las características de esta adherencia dependen de mecanismos como la adherencia química entre acero y concreto, la fricción generada entre los materiales, así como procedimientos mecánicos de transferencia de fuerza proporcionados por las corrugaciones del acero de refuerzo. Al usar acero no corrugado, el único mecanismo es la adherencia química, provocándose un deslizamiento temprano del acero respecto al concreto y la imposibilidad de alcanzar mayor resistencia que la de deslizamiento. La fuerza de adherencia al emplear acero corrugado, como se indica en la Fig.17, se genera por el mecanismo entre la corrugación y el concreto circundante. Las dimensiones de la corrugación en esta clase de acero, determina el tipo de falla en el elemento si este falla por problema de adherencia. Así, si la corrugación es de poca altura se podrá generar una falla por aplastamiento en el concreto. Por otro lado, si la corrugación tiene gran altura, esta podrá fallar por flexión. Igualmente, si la separación entre las corrugaciones tiende a ser grande, la resistencia por adherencia tendera a disminuir. Es por eso que en las normas y códigos existe o debe existir una normatividad respecto a alturas máxima y mínima de la corrugación y valores máximos y mínimos de separación entre corrugaciones. Dos tipos comunes de falla por adherencia y anclaje, dependientes de las características geométricas de las barras de refuerzo corrugadas, se presentan en la Fig.18.

Los factores determinantes en la resistencia por adherencia y anclaje de un elemento de concreto reforzado son los siguientes:

a) Resistencia del concreto: Debido al estado de esfuerzos a que se somete el concreto en la vecindad del acero, a mayor resistencia a tensión del concreto la resistencia por adherencia será mayor.

b) Características dimensionales del acero de refuerzo: Como se explicó antes, el uso del acero corrugado, por el mecanismo que se genera entre la corrugación y el concreto, provoca aumento en la resistencia por adherencia. El hecho que en barras de menor diámetro se obtenga mayor resistencia por adherencia provoca preferencia por el uso de barras de diámetros pequeños.

c) Posición y orientación del acero de refuerzo: La resistencia por adherencia en aceros colocados verticalmente resulta mayor que para aceros colocados horizontalmente. También, por fenómenos de sedimentación de los agregados es mas común encontrar formación de burbujas de aire en el concreto de la vecindad del refuerzo superior de una viga, provocando que la resistencia por adherencia en el acero de refuerzo inferior sea aproximadamente 20% mayor que la obtenida en el acero de refuerzo superior.

d) Dimensión del recubrimiento: La resistencia por adherencia será mayor a mayor dimensión del recubrimiento. Esto es debido a que el peso propio del recubrimiento y la superficie del concreto que estará sujeto a estado de esfuerzos de tensión son mayores a mayor recubrimiento.

e) Configuración y distribución del acero de refuerzo lateral: El acero de refuerzo lateral juega un factor importante para evitar el rápido incremento de la abertura del agrietamiento por adherencia (paralelo al acero de refuerzo longitudinal), contribuyendo con ello a incrementar la resistencia y capacidad de transmitir fuerza por efecto de adherencia. El acero de refuerzo lateral no tiene efecto en impedir la aparición del agrietamiento por adherencia o efecto de dovela del acero longitudinal. Sin embargo, posterior al agrietamiento contribuye a que el decaimiento o degradación de la resistencia por adherencia sea menor (Fig.19).

Las limitaciones presentadas en los códigos y reglamentos respecto al uso de paquetes de barras de refuerzo longitudinal, separación mínima entre las mismas y dimensiones mínimas de recubrimiento, entre otras condicionantes, están fundamentadas en la necesidad de impedir degradación o decaimiento de la resistencia por adherencia y anclaje, para garantizar que se alcanzará la resistencia última del elemento en particular y la estructura en general, resistencia última que fue propuesta en la etapa de diseño.

6. Comportamiento de elementos viga y columna.

6.1 Factores que determinan el mecanismo de falla.

Como factores importantes en la resistencia y capacidad de deformación de elementos estructurales de concreto reforzado lineales (vigas y columnas), se pueden proponer los siguientes:

a) Cantidad y diámetro del acero de refuerzo longitudinal.

Al colocar la misma cantidad de acero de refuerzo, pero de menor diámetro, se incrementa la superficie de contacto acero-concreto y por lo tanto se incrementa la resistencia por adherencia y anclaje. Sin embargo, existen límites en el tamaño mínimo del acero de refuerzo longitudinal debido a que, como resultado de trabajos experimentales, el tamaño de las corrugaciones en barras de diámetros menores a 19 mm no resultan lo suficientemente eficientes para la transmisión mecánica de fuerza de adherencia.

b) Cantidad y distribución del acero de refuerzo lateral.

Principalmente, como resultado de trabajos experimentales recientes, se entiende que el papel del acero de refuerzo lateral en elementos de concreto reforzado, además de contribuir a evitar una falla frágil por cortante en el elemento, también tiene efecto sobre las siguientes características de un elementos estructural:

- Proporciona confinamiento al concreto del núcleo en los elementos lineales (principalmente elementos columna). Incrementando la resistencia a la compresión del concreto del núcleo y también aumentando el valor de la deformación unitaria última, lo que contribuye a mejorar la capacidad de deformación del elemento.
- Evitar la falla por adherencia y anclaje. Para lo que se recomienda que, para iguales cantidades de acero de refuerzo lateral, la separación del mismo sea la menor posible y que, de ser posible, las barras de refuerzo longitudinal estén confinadas directamente por una esquina o un gancho de dicho refuerzo lateral. Esto incrementará la capacidad y resistencia del refuerzo longitudinal por adherencia y anclaje notablemente.
- Proporciona soporte lateral al acero de refuerzo longitudinal, evitando de esta manera el pandeo del mismo. Logrando un adecuado soporte lateral en el acero de refuerzo longitudinal, también se está contribuyendo a que este participe como confinante del concreto del núcleo del elemento.

c) Efecto de elementos vecinales, como losa de piso y trabes ortogonales.

De investigaciones experimentales recientes, se ha concluido que una losa estructural, reforzada y anclada adecuadamente al elemento viga correspondiente durante el proceso constructivo, participa totalmente junto con la viga en rigidez y resistencia en el trabajo del marco momento resistente. Esto al alcanzar el elemento y/o la estructura su resistencia última.

Igualmente, el efecto de elementos ortogonales es de gran importancia sobre todo al realizar, para el diseño de la estructura, análisis planos. Se ha comprobado experimental y analíticamente que tanto en estructuras a base de muros estructurales, como en aquellas a base de marcos momento resistentes, la rigidez y resistencia obtenida de un análisis plano es notablemente menor que la real. Esto no necesariamente contribuye a incrementar el factor de seguridad de la estructura, ya que puede generar cargas axiales a niveles indeseables en columnas y muros, así como efectos de torsión y cortante en vigas que no fueron contemplados en el análisis plano.

d) Carga axial.

Para lograr incrementos en la capacidad de deformación en elementos sujetos a efectos principales de carga axial y flexión (columnas), como producto de análisis teórico-experimentales se recomiendan contemplar los siguientes aspectos:

- Reducir la carga axial suficientemente bajo la carga axial del estado de esfuerzos "balanceado"

$$N / (\sigma_B \times b \times D) \leq 0.3$$

- Incrementar la cantidad de refuerzo longitudinal a compresión.
- Incrementar el confinamiento en el concreto del núcleo, con refuerzo lateral (espirales, ganchos, estribos, etc), en las secciones críticas a flexión (vgr. aquellas donde se prevé la formación de articulaciones plásticas, principalmente en las todas las vigas y las columnas del primer nivel). En la Fig.20 se presentan algunos ejemplos de refuerzo lateral en columnas básicos para lograr comportamientos adecuados de los elementos.
- Reducir los esfuerzos por cortante al alcanzar la resistencia por flexión, a límites como el indicado:

$\tau_u \leq 30$ kgf/cm^2 (τ_u representa al esfuerzo cortante en la sección transversal)

e) Carga cíclica.

El efecto de carga cíclica provoca efectos, a largo plazo, similares a loa provocados por problemas de fatiga en los materiales. A mayor el número de ciclos, mayor será la degradación del material (el concreto en este caso), generando disminución de la capacidad de deformación y decaimiento de resistencia en el rango posterior a la fluencia.

6.2 Misceláneos.

a) En la mayoría de los reglamentos para la construcción, en cuanto a estructuras de concreto reforzado, se acepta el "corte" del acero de refuerzo longitudinal después de asegurar una "longitud de desarrollo" adecuada. Es recomendable, en estos casos, emplear doblez en los extremos de las barras, con el propósito de lograr una buena transmisión de los esfuerzos de adherencia hacia el concreto (Fig.21)

b) Por limitaciones de fabricación y/o transporte, el acero se adquiere a longitudes fijas, es por tal motivo que cuando se requiere proporcionar longitudes de acero refuerzo longitudinal en vigas o columnas de mayor longitud que las barras adquiridas en fábrica se hace uso del traslape de acero de refuerzo. En los reglamentos basados en información experimental basta (como es el caso del reglamento del AIJ), se permite el empleo de traslapes mecánicos (soldadura a presión y gas, tuercas de alta resistencia, splice con relleno de mortero de alta resistencia, etc., algunos de los cuales se presentan en la Fig.22) en la misma sección. Esto debido a que se ha comprobado experimentalmente que si el traslape mecánico es de calidad comprobada por pruebas a tensión, es prácticamente improbable la formación de una superficie de falla en la sección en cuestión.

Cuando el traslape es convencional, se recomienda seguir la tendencia de no realizarlos en la misma sección del elemento estructural.

c) Finalmente, en las Fig.23 y Fig.24 se ejemplifican gráficamente fallas típicas en vigas y columnas. En la Fig.25 se muestra una viga con un detalle de refuerzo típico para un agujero en el alma propio de requisitos para instalaciones. En la Fig.26 se presentan detalles de refuerzo adecuados para una viga con desnivel en el interin del claro y el detalle de la llegada de una viga secundaria apoyándose en una primaria.

7. Comentarios sobre el diseño de estructuras de cimentación.

Para diseñar la estructura de cimentación se deberá considerar todos los posibles estados de carga que pudiera sufrir la misma, como el estado de cargas verticales y horizontales generadas por de la formación de mecanismo de falla ante el sismo de diseño, así como la posible situación de una descarga del inmueble por reparación o remodelación que pudiera repercutir en asentamientos diferenciales o emersión de la subestructura. De igual manera, se deberá considerar y diseñar adecuadamente la zona de unión entre estructura y subestructura, poner especial atención en los anclajes de los elementos verticales (columnas y muros) en las correspondientes estructuras de cimentación.

Al diseñar una estructura, dependiendo del reglamento a emplear, se permite la formación de los llamados mecanismos de fluencia en la superestructura. Sin embargo, en el caso de las estructuras de cimentación, como regla general, se prohíbe la formación de articulaciones plásticas, por lo que todos los elementos de la estructura de cimentación deberán diseñarse con la filosofía de diseño elástico.

Cuando en la superestructura se consideren muros estructurales de cortante, la contratrabe de cimentación deberá diseñarse para ser lo suficientemente rígida y resistente para soportar las rotaciones del muro al trabajar este como cantiléver al formarse un mecanismo de fluencia con aparición de articulaciones plásticas en trabes.

En el caso de zapatas "aisladas", es recomendable el considerar su liga por medio de contratrabes de liga con rigidez y resistencia adecuada para soportar y reducir los efectos producto de asentamientos diferenciales.

Cuando la cimentación es con pilotes (de punta o de fricción), el pilote se diseñará para soportar desde las fuerzas generadas por si hincado (como es el impacto axial), fuerzas de tensión provocadas por el momento de volteo de la estructura global o por fricción negativa, en este tipo de cimentaciones se revisara el anclaje en la junta del pilote con el dado de cimentación. Cuando no se use dado de cimentación, se revisara la posibilidad de penetración entre pilote y losa de cimentación. Con respecto a cimentación a base de pilotes y dado, se tendrá cuidado en no debilitar al pilote en su cabezal o en su llegada al dado, por la diferencia de rigidez y resistencia entre ambos elementos, por lo que es recomendable proporcionar un refuerzo especial en la zona del cabezal del pilote. En este tipo de cimentación, también se consideraran todos los posibles elementos mecánicos que surgen a lo largo del pilote para el diseño del mismo y de las juntas entre los segmentos.

Finalmente, es evidente que para lograr un diseño confiable de la estructura de cimentación, el diseñador deberá contar con la información precisa y confiable de la mecánica del suelo del sitio de construccion (cuyos niveles de precisión y confiabilidad dependerán del tipo de estructura).

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FIGURAS

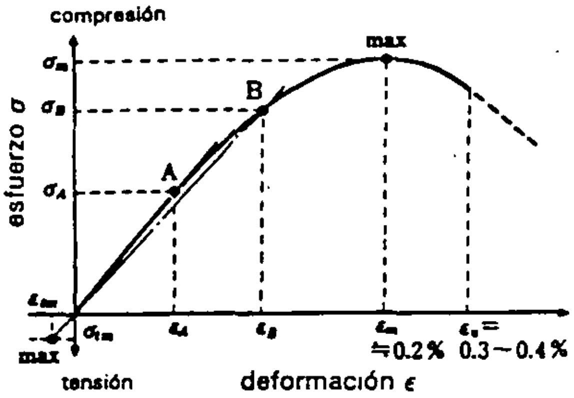


Fig. 1 Curva esfuerzo deformación de concreto normal

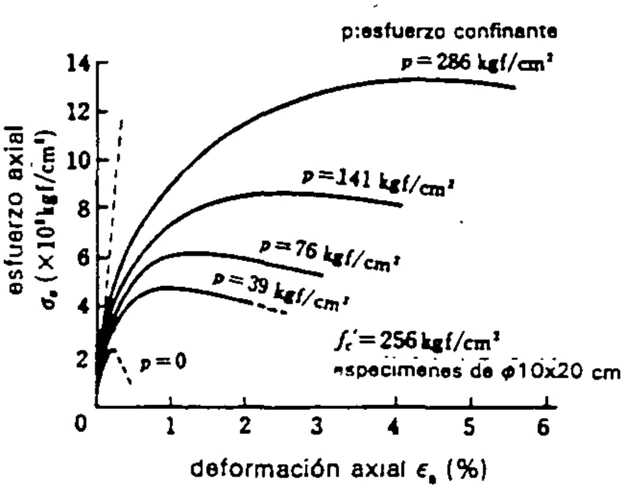
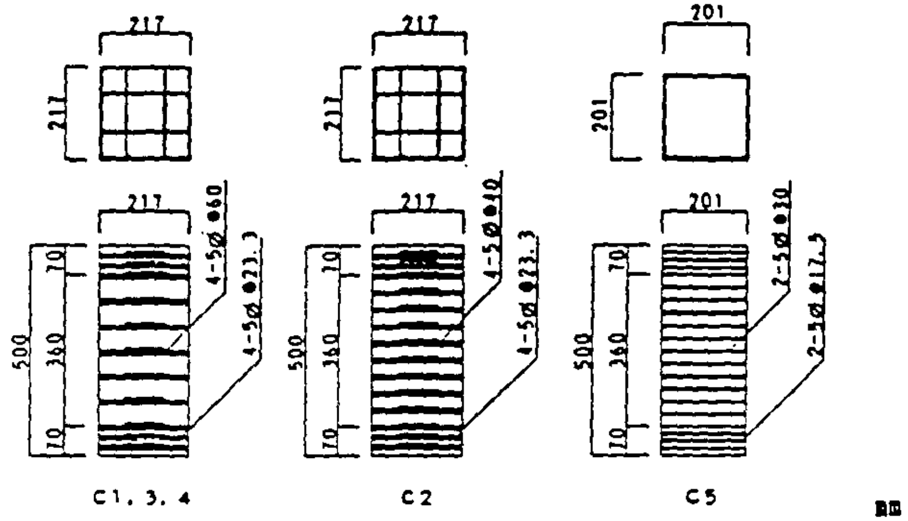


Fig. 2 Curva deformación de concreto con confinamiento lateral



especímenes empleados

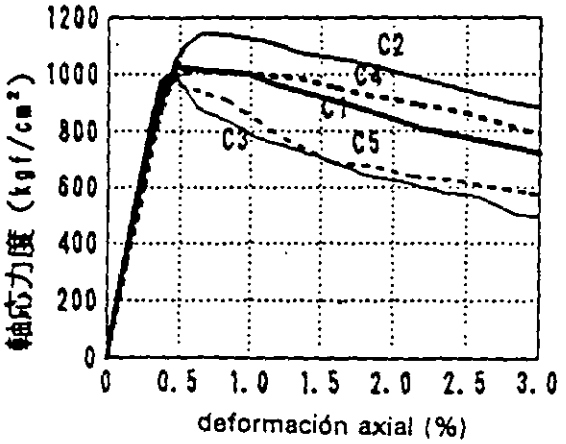


Fig. 3 Relación esfuerzo deformación de elementos cargados axialmente a compresión

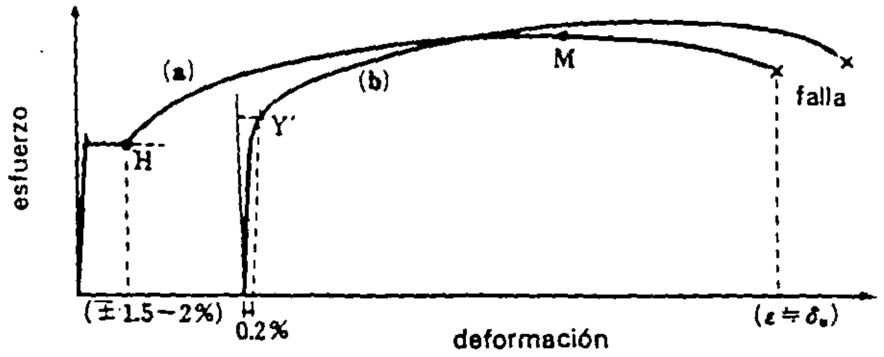


Fig. 4 Relación esfuerzo deformación real del acero

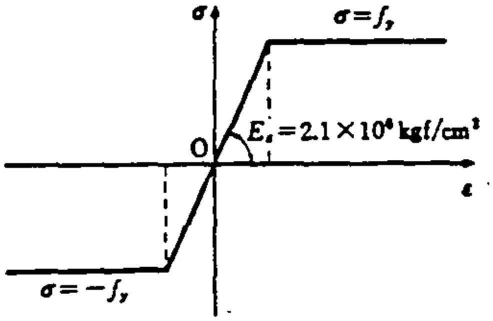


Fig. 5 Relación esfuerzo deformación idealizada del acero

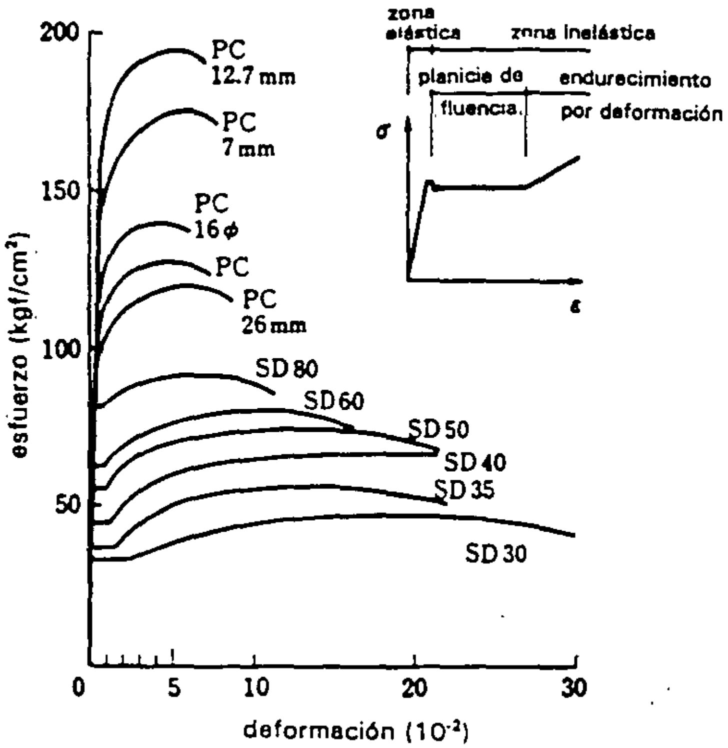
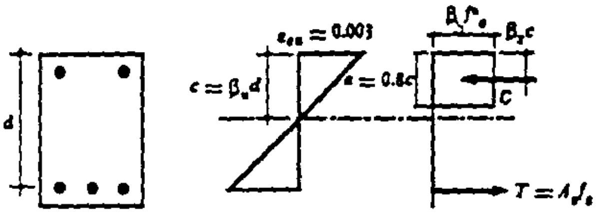


Fig. 6 Relaciones esfuerzo-deformación para barras de refuerzo de diferentes características



$\beta_2 = 0.40$ para sec. rectangulares

$$\beta_1 = \left(1.05 - \frac{f_c^*}{1\,400}\right) \leq 0.85$$

$f_s \leq f_y^*$

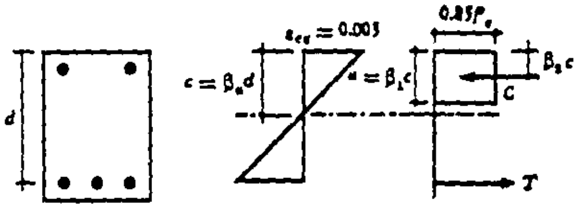
$f_c^* = \alpha f'_c (1 - v)$

$f_y^* = \gamma f_y$

v = coeficiente de variación del concreto

α y γ = coeficientes que dependen del tipo de carga

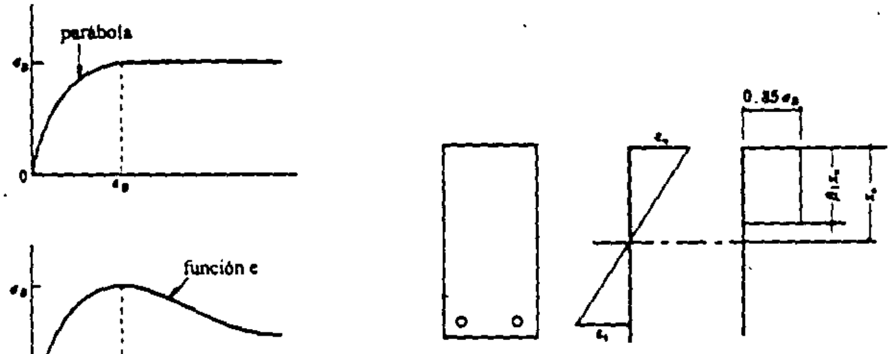
(a) RDF



$$\beta_1 = \left(1.05 - \frac{f'_c}{1\,400}\right) \leq 0.85$$

f'_c en kg/cm²

(b) ACI



Relación $\sigma - \varepsilon$ normalmente empleado en el concreto

$\beta_1 = 0.85$ [$\sigma_B \leq 280$ kgf/cm²]

$\beta_1 = 0.85 - 0.005\ (\sigma_B - 280)/70$ [$\sigma_B \geq 280$ kgf/cm²]

Xn: posición del eje neutro a calcular

(c) AIJ

Fig. 7 Hipótesis simplificatorias para el ánalisis por flexión según los reglamentos RDF, AIC y AIJ

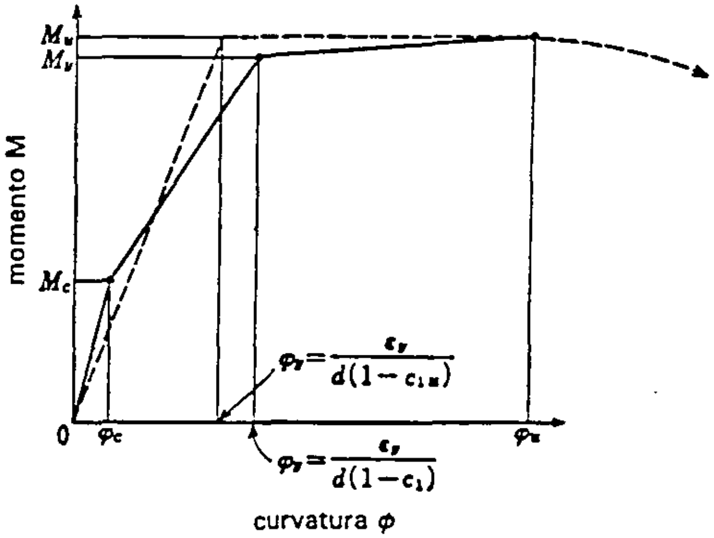


Fig. 8 Diagrama momento-curvatura hipotetizado

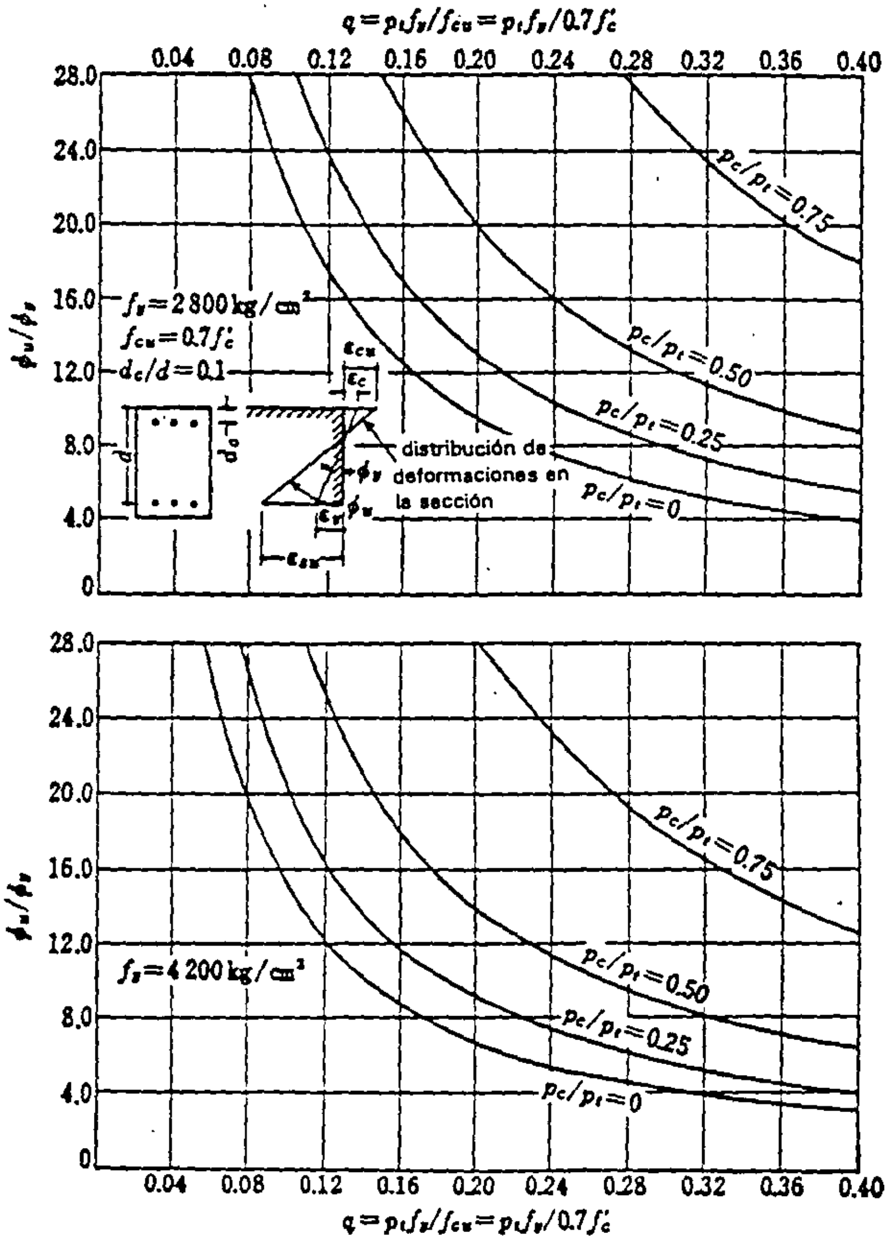


Fig. 9 Ductilidad calculada de la sección transversal de una viga

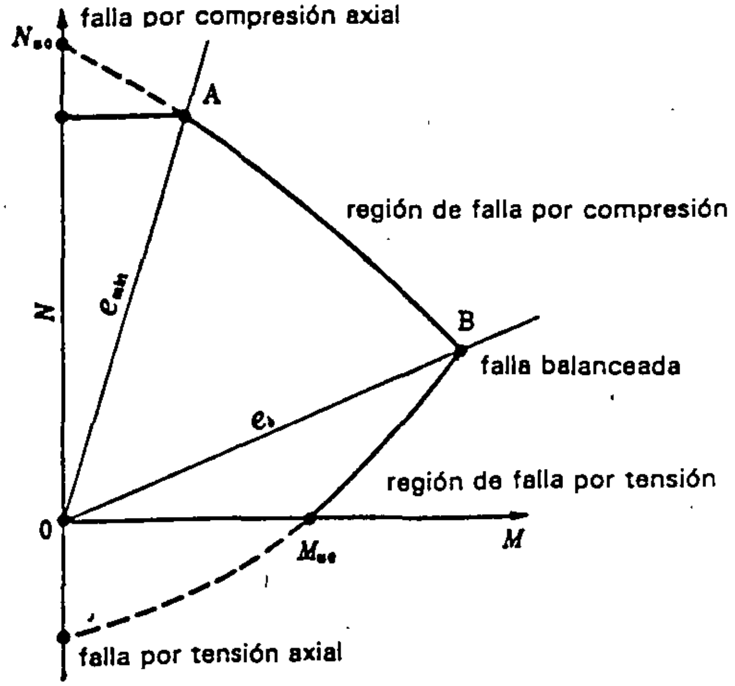


Fig. 10 Diagrama de interacción carga axial (N) y momento flexionante (M)

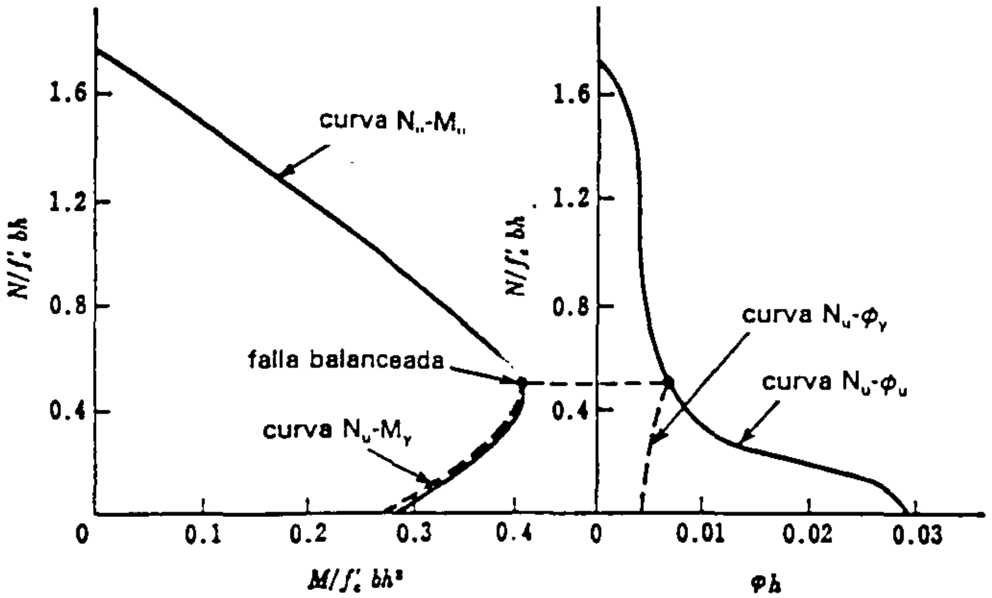
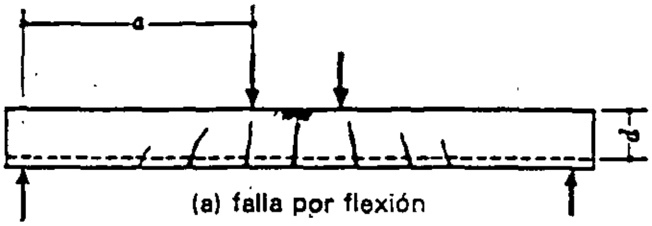
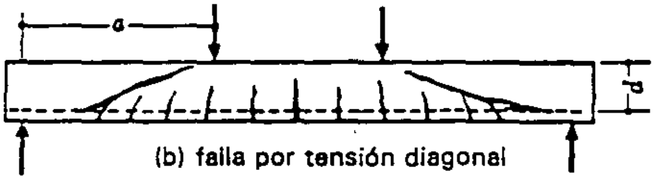


Fig. 11 Gráfica de carga axial (N), momento (M) y curvatura (ϕ)

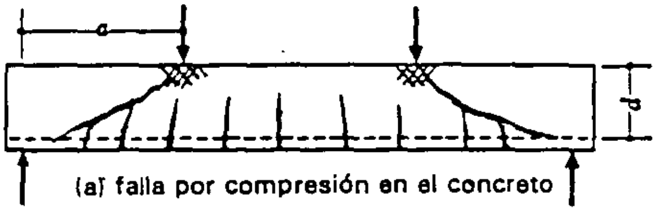


(a) falla por flexión

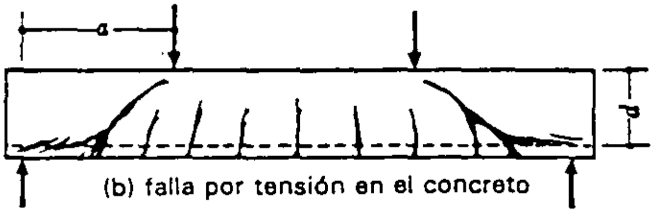


(b) falla por tensión diagonal

Fig. 12 Representación de la falla por cortante en vigas de relación a/d grande



(a) falla por compresión en el concreto



(b) falla por tensión en el concreto

Fig. 13 Representación de la falla por cortante en vigas de relación a/d pequeña

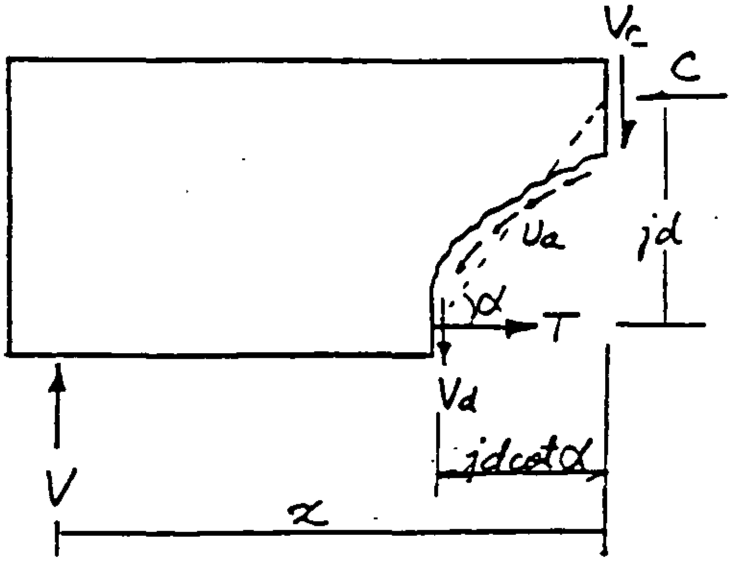


Fig. 14 Equilibrio en el claro de cortante de la viga (sin refuerzo lateral)

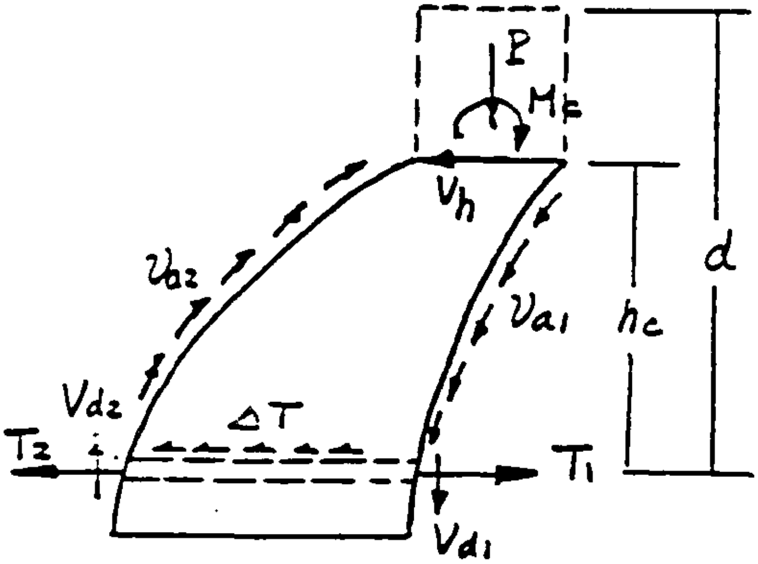


Fig. 15 Acciones en el "cantiléver" formado por grietas de flexo-cortante

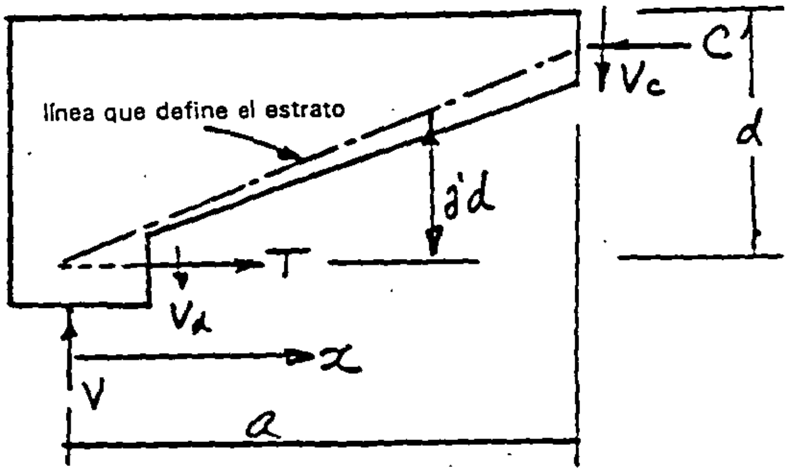


Fig. 16 Mecanismo de arco de transmisión de fuerza cortante

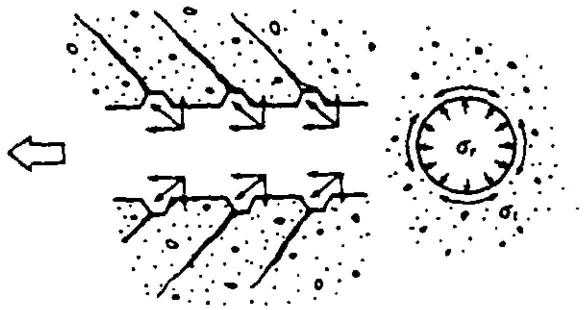
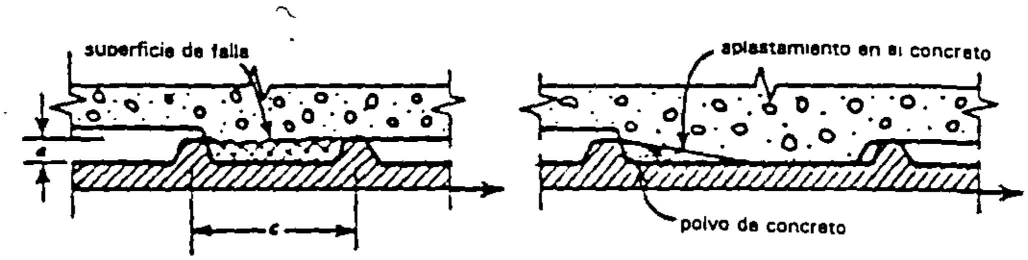


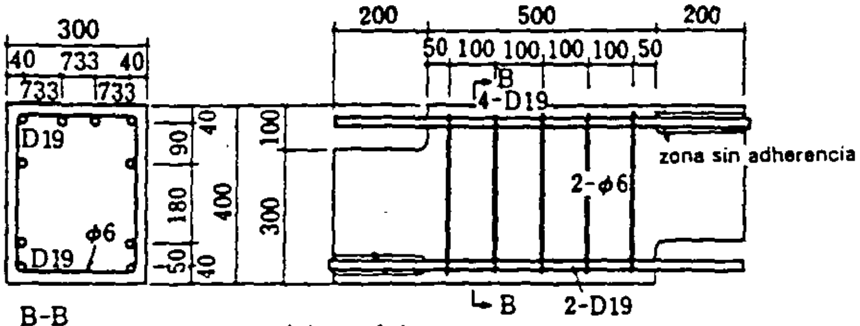
Fig. 17 Cortante directo o cizalleo entre el acero corrugado y el concreto circundante



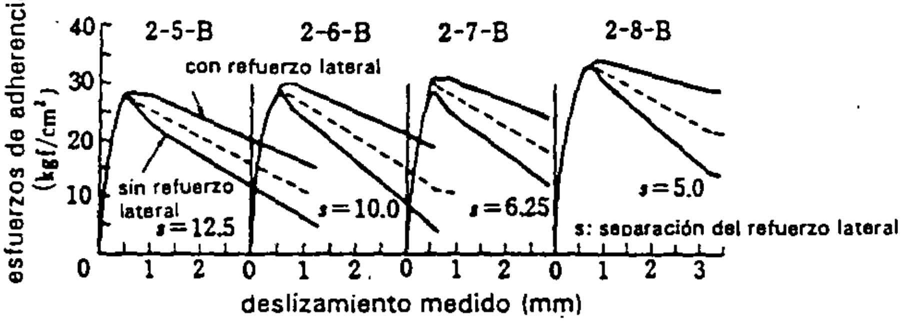
(a) falla por cortante directo (a/c > 0.15)

(b) falla de anclaje (a/c < 0.10)

Fig. 18 Dos mecanismos de falla representativos en el problema de adherencia y anclaje

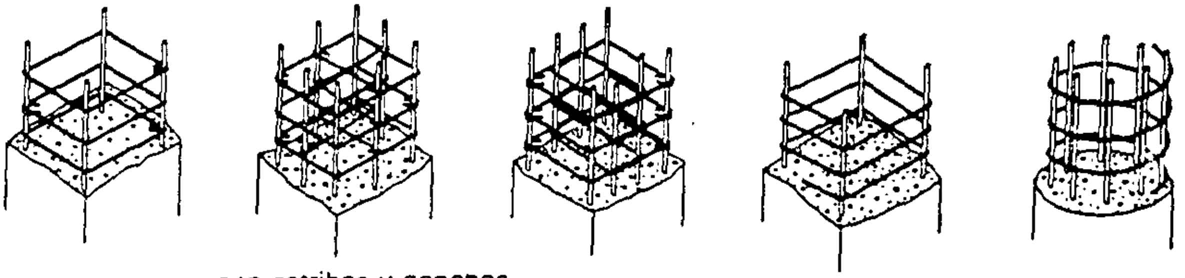


(a) espécimen



(b) Relación entre esfuerzos de adherencia y deslizamiento

Fig. 19 Resultados experimentales sobre adherencia y anclaje en acero corrugado



con estribos y ganchos

con espiral

Fig. 20 Distribución de refuerzo lateral típico en columnas

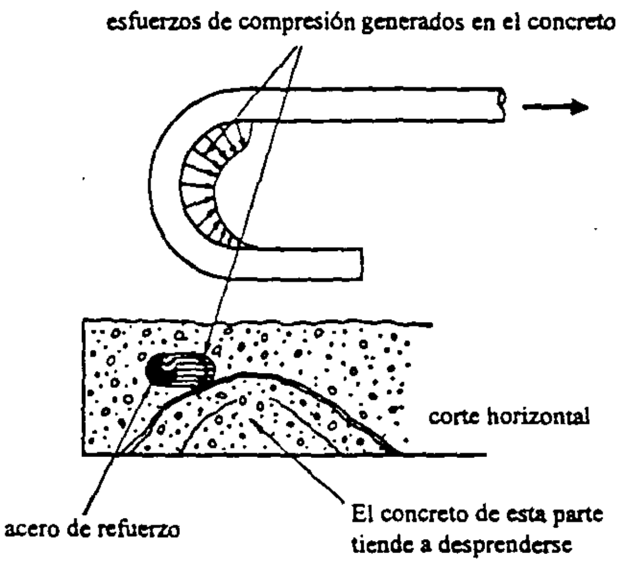


Fig. 21 Efecto del gancho en el traslape de refuerzo

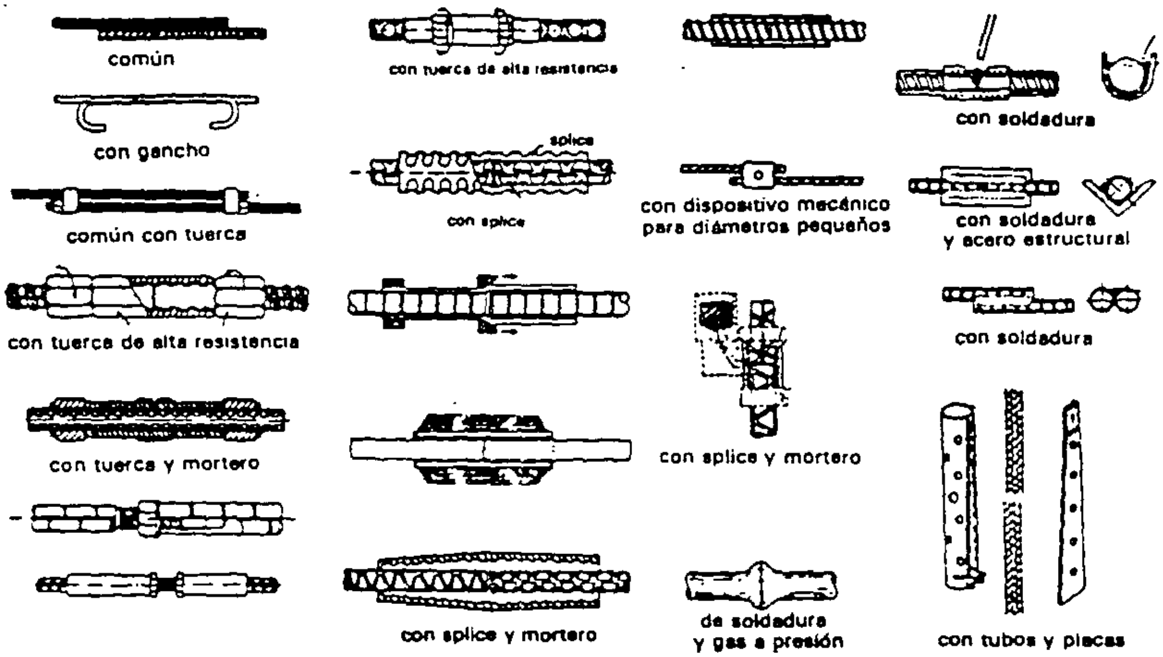
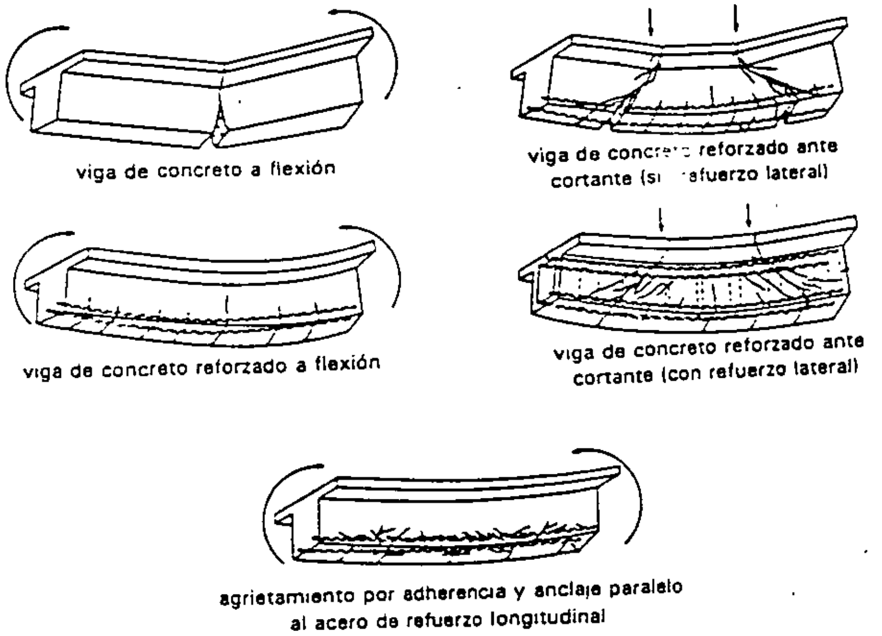


Fig. 22 Diversos métodos de traslape en acero de refuerzo



viga de concreto a flexión

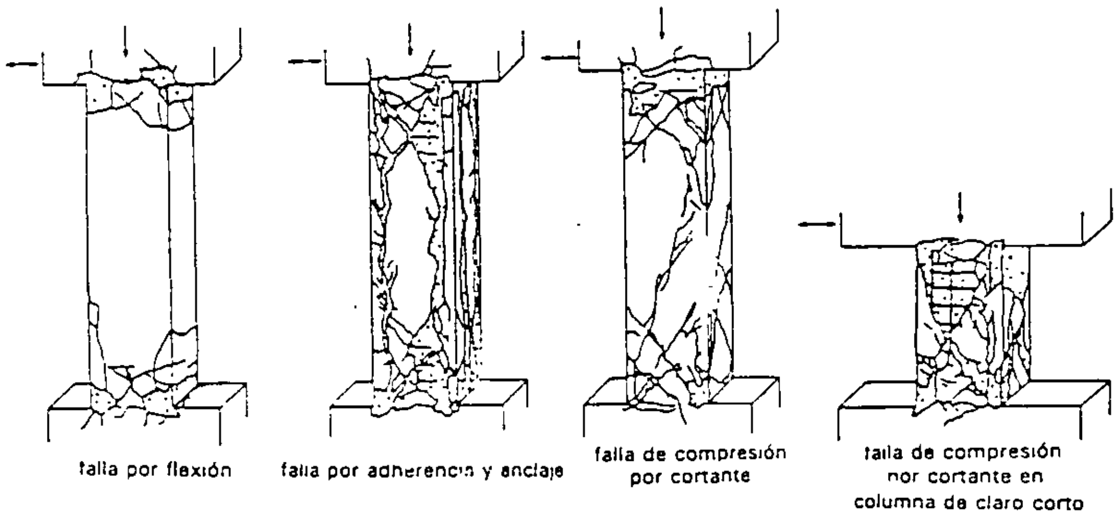
viga de concreto reforzado ante cortante (sin refuerzo lateral)

viga de concreto reforzado a flexión

viga de concreto reforzado ante cortante (con refuerzo lateral)

agrietamiento por adherencia y anclaje paralelo al acero de refuerzo longitudinal

Fig. 23 Modos de falla en vigas



falla por flexión

falla por adherencia y anclaje

falla de compresión por cortante

falla de compresión por cortante en columna de claro corto

Fig. 24 Modos de falla en columnas

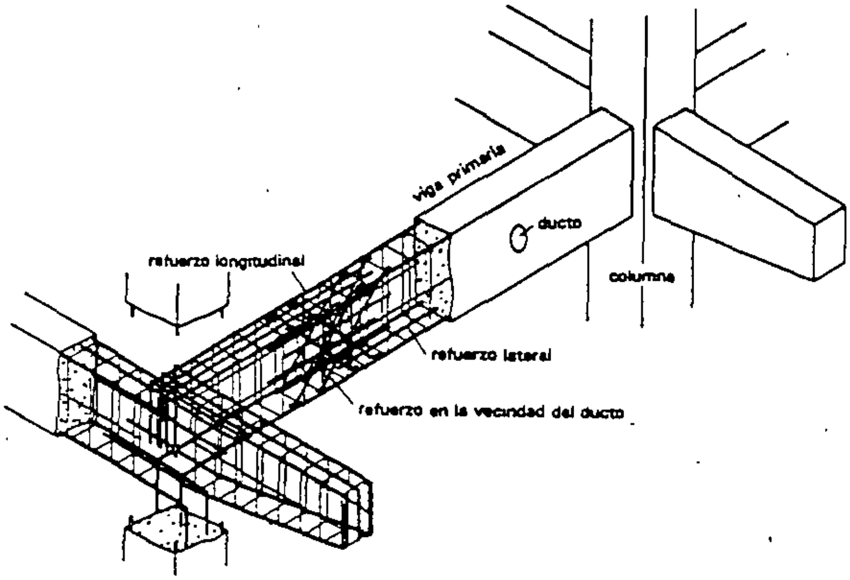


Fig. 25 Detalle de refuerzo para ductos en el alma de viga

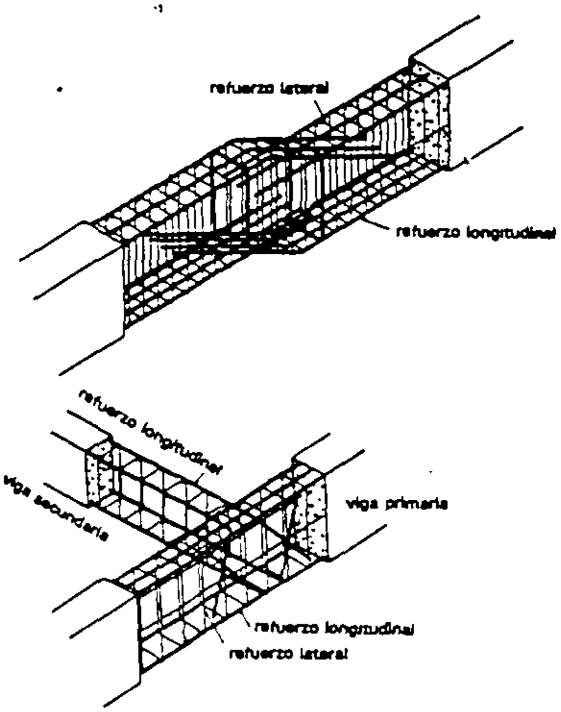


Fig. 26 Detalle de refuerzo en viga con desnivel y del apoyo
de una viga secundaria en una primaria

COMPORTAMIENTO Y DISEÑO DE ESTRUCTURAS DE CONCRETO REFORZADO

MUROS ESTRUCTURALES

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1. INTRODUCCION

Es común que se denomine a los muros de concreto reforzado como "muros de corte" o "muros de cortante" porque resisten un alto porcentaje de la fuerza cortante lateral total. Sin embargo, estos términos son desafortunados y un tanto engañosos puesto que la mayoría de los muros se pueden diseñar de manera que tengan un comportamiento dominado por flexión, y que, por tanto, exhiban un modo de falla dúctil. En este capítulo usaremos el término "muros estructurales de concreto" para referirnos a los muros que deberán resistir las fuerzas inducidas por las aceleraciones sísmicas.

Los muros estructurales bien diseñados y detallados ofrecen varias ventajas para su uso en zonas sísmicas:

1. Poseen una mayor rigidez que la de marcos de concreto reforzado.
2. Dada su alta rigidez, exhiben un comportamiento adecuado ante sismos moderados.
3. Poseen una buena capacidad de deformación (ductilidad) que les permite resistir sismos intensos.

Los muros estructurales deben diseñarse para resistir la variación del cortante en la altura (que es máximo en la base), del momento, que produce compresión en un extremo y tensión en el extremo opuesto, así como las cargas gravitacionales que producen compresión en el muro (Fig. 1). La cimentación debe diseñarse para resistir el cortante y el momento máximos que pueden desarrollarse en la base del muro. El refuerzo en la base debe detallarse cuidadosamente para que las fuerzas puedan transferirse entre el muro y la cimentación; en particular, se debe enfatizar la unión y el anclaje de varillas.

Aunque es difícil satisfacer todos los requisitos de funcionamiento de un edificio, los muros estructurales deben colocarse de manera que la distribución de rigidez en planta sea simétrica y que la configuración sea estable torsionalmente (Fig. 2). Además se debe observar que la cimentación pueda resistir el momento de volteo de la base. Es preferible la colocación de un mayor número de muros estructurales en el perímetro como sea posible. Otro aspecto a considerar es que mientras mayor sea la carga gravitacional resistida por un muro, menor será la demanda por refuerzo de flexión y más facil será la transmisión de momentos de volteo a la cimentación. Por tanto, a menor cantidad de muros, mayores son las fuerzas que deben ser transmitidas a la cimentación.

2. TIPOS DE MUROS ESTRUCTURALES

2.1 Según la Forma de su Sección Transversal

Atendiendo a la sección transversal los muros pueden ser como los presentados en la Fig. 3. En algunas ocasiones los muros poseen elementos extremos (Figs. 3b, 3c, 3d) para permitir el anclaje adecuado de vigas transversales, para colocar el refuerzo a flexión, para dar estabilidad a muros con almas angostas y para proporcionar un confinamiento más efectivo del concreto en la zona de articulación plástica.

Por lo general, el espesor mínimo de un muro estructural es de 20 cm si se emplean varillas corrugadas para su refuerzo, y de 15 cm si se usa malla de acero electrosoldada.

2.2 Según su Forma en Elevación

La mayor parte de los muros son prismáticos, es decir, que no sufren cambios de dimensiones en elevación. Sin embargo es frecuente que su espesor disminuya con la altura. De acuerdo con las variaciones en la altura, los muros estructurales se pueden clasificar como muros estructurales sin aberturas y muros con aberturas. En el último caso las aberturas se dejan para colocar ventanas o puertas o ambas.

La mayoría de los muros estructurales sin aberturas se puede tratar como una viga-columna. Las fuerzas laterales son introducidas mediante una serie de cargas puntuales a través de los diafragmas de piso. Dada su relación de aspecto altura del muro / longitud h_w/l_w, se distinguen muros esbeltos con relaciones h/l mayores que dos, y muros robustos para relaciones menores o iguales a dos (Fig. 4). Es importante señalar que los muros bajos (robustos) poseen una elevada resistencia a la flexión, aun para refuerzo vertical mínimo, por lo que es necesario aplicar fuerzas cortantes muy altas para desarrollar dicha resistencia. Esto provoca que el comportamiento de este tipo de muros sea dominado por corte.

Las aberturas de los muros deben colocarse de forma que no disminuyan las resistencias a la flexión y al cortante. Un ejemplo de ello es la Fig. 5a. Si las aberturas se colocan de manera alternada en elevación es recomendable la colocación de refuerzo diagonal para ayudar en la formación de campos diagonales a compresión y a tensión una vez que el muro se ha agrietado diagonalmente (Fig. 5b). Si las aberturas se colocan en forma regular se obtiene un tipo de muros llamados acoplados que poseen excelentes características de comportamiento sísmico (Fig. 6).

2.3 Según su Comportamiento

Según su comportamiento, los muros estructurales de concreto se pueden dividir en:

1. Muros de cortante, en los cuales el corte controla las deflexiones y la resistencia;
2. Muros de flexión, en los que la flexión controla las deflexiones y la resistencia;
3. Muros dúctiles (muro estructural "especial") que poseen buenas características de disipación de energía ante cargas cíclicas reversibles.

Si esperaramos un comportamiento esencialmente elástico, cualquier tipo de muro de los arriba citados sería adecuado. Sin embargo, si anticipamos que el muro estará sometido a deformaciones en el intervalo inelástico, como ante sismos, es inaceptable el uso de muros de cortante; es preferible un muro dúctil.

3. MUROS ESTRUCTURALES ESBELTOS

3.1 Modos de Falla y Criterio de Diseño

Un prerrequisito para el diseño de muros estructurales dúctiles es que la fluencia del refuerzo de flexión en zonas de articulación plástica definidas controle la resistencia, las deformaciones inelásticas y la capacidad de deformación de toda la estructura. De esta manera, la principal fuente de disipación de energía será la plastificación del acero a flexión (Fig. 7b y 7e). Se deben evitar los modos de falla debidos a la fractura del acero a flexión (Fig.

7f), a tensión diagonal (Figs. 7c y 7g) o a compresión diagonal causados por cortante (Fig. 7h). Asimismo, se deben evitar las fallas causadas por inestabilidad del alma del muro o del refuerzo principal a compresión, el deslizamiento por cortante a lo largo de juntas de construcción (Fig. 7d) y la falla por cortante o ahderencia a lo largo de uniones de barras o de anclajes.

En la Fig. 8 se muestra la respuesta histerética de un muro estructural controlada por la resistencia al corte. Es evidente la continua reducción en la resistencia y en la capacidad de disipación de energía con los ciclos. Por el contrario, en la Fig. 9, se presenta la respuesta histerética estable de un muro estructural dúctil. Es claro que aun para una ductilidad de desplazamiento igual a cuatro, la respuesta exhibe una capacidad de disipación de energía muy buena. El comportamiento de muros estructurales dúctiles es comparable con el de columnas; su capacidad de rotación plástica es afectada por fuerzas axiales y cortantes.

Puesto que el área bruta de la sección de un muro estructural es muy grande, las cargas axiales que obrarán sobre él estarán muy por debajo del punto balanceado; debido a lo anterior, una adecuada ductilidad de curvatura se logrará si:

1. Se coloca el refuerzo por flexión en los extremos del muro; y
2. Se confinan estos extremos mediante estribos con bajas separaciones. El confinamiento aumentará la capacidad de deformación útil del concreto y retrasará el pandeo del acero de flexión.

Para evitar problemas de corte, el diseño de flexión debe garantizar que:

1. El agrietamiento diagonal del muro no ocurra aun ante los momentos máximos que se pueden producir por el muro;
2. Si ocurriese el agrietamiento diagonal, el cortante sería resistido por el refuerzo del muro, y
3. Los esfuerzos nominales de corte deben mantenerse bajos para retrasar la falla por deslizamiento del muro y para prevenir el aplastamiento del concreto en el alma.

Los criterios de diseño escritos arriba son fácilmente satisfechos en muros esbeltos cuyo comportamiento por naturaleza es dominado por flexión. Sin embargo, es prácticamente imposible diseñar los muros robustos para que su comportamiento sea dominado por flexión. En esos casos es preferible diseñar los muros para que permanezcan elásticos ante las cargas máximas anticipadas.

3.2 Resistencia a la Flexión

Para diferentes cargas axiales en los muros es factible calcular la relación momento-curvatura empleando un programa de computadora. Como se mencionó en el capítulo sobre columnas, este diagrama describe el comportamiento del elemento; es similar a un diagrama esfuerzo-deformación de un material. Afortunadamente, los diagramas se pueden obtener en forma aproximada empleando métodos simples tales como los usados para columnas.

Suponiendo un bloque equivalente de esfuerzos en el concreto a compresión y que el acero a tensión está sometido a un esfuerzo igual o menor que el esfuerzo de fluencia, se puede obtener por equilibrio de fuerzas en la sección que la capacidad a flexión de un muro estructural está dada por la Ec. 1.

$$M_n = 0.5 A_s f_y l_w \left(1 + \frac{N_n}{A_s f_y}\right) \left(1 - \frac{c}{l_w}\right) \qquad \textbf{(1)}$$

donde A_s es el área de acero a tensión en el muro (refuerzo vertical);
f_y es el esfuerzo de fluencia del acero vertical de los muros;

l_w es la longitud del muro;
N_u es la carga axial actuante; y
c es la profundidad del eje neutro medida desde la fibra a compresión máxima.

Si continuamos con la analogía de muros estructurales esbeltos con columnas, es claro que se puede obtener una mayor resistencia a la flexión si concentramos el refuerzo vertical (a flexión) en las fibras extremas de la sección transversal. En la Fig. 10 se presenta la comparación del comportamiento de dos muros esbeltos con misma cantidad de refuerzo por flexión, pero en donde en uno de los muros el refuerzo está distribuido uniformemente en la longitud del muro y en el otro se ha concentrado en los extremos, manteniendo solamente refuerzo mínimo en la porción intermedia. De la gráfica se puede concluir que los muros con refuerzo concentrado en los extremos son, en comparación con aquellos con refuerzo distribuido, más resistentes y mucho más dúctiles. Este incremento en la eficiencia, sin embargo, se puede ver contrarrestada si el acero a flexión alcanza deformaciones dentro del intervalo de endurecimiento de deformación ya que la ductilidad disminuye. Es necesario entonces confinar los elementos extremos de los muros en donde se concentra el acero (ver sección 3.4).

Si colocamos el refuerzo por flexión en el muro en cantidad igual a la requerida por el momento flexionante obtenido del análisis de la estructura, es teóricamente posible la formación de la articulación plástica en cualquier parte de la altura del muro. Por tanto, si deseamos que la articulación se forme en la base del elemento es necesario diseñar por flexión el resto del muro por arriba del momento último (sobrediseñar). Además, el refuerzo por flexión debe cortarse de manera que la articulación ocurra en la base. Con base en información experimental, la longitud de la articulación plástica sobre la altura del muro varía entre $0.3l_w$ y $0.8l_w$. Fuera de esta región, las varillas deberán tener una longitud igual ala longitud de desarrollo.

3.3 Resistencia al Cortante

La resistencia al corte en muros estructurales esbeltos está proporcionada por el concreto y el acero horizontal. El componente de la resistencia debida al concreto depende de que hayan aparecido grietas diagonales en el alma del muro o que el muro exhiba fisuras por flexión-cortante. En el primer caso, las grietas empiezan cerca del centro del alma y aparecen cuando los esfuerzos principales a tensión exceden a la resistencia a tensión del concreto.

Para fines de diseño, la contribución del concreto a la resistencia se puede tomar de manera conservadora igual a la empleada en vigas. En el reglamento para estructuras de concreto del Instituto Americano del Concreto, se presentan dos expresiones alternas para calcular esta contribución; sin embargo, las Normas Técnicas Complementarias para Diseño y Construcción de Estructuras de Concreto para el Distrito Federal (NTC-Concreto) sólo consideran la expresión para vigas.

La contribución del refuerzo horizontal a la resistencia a fuerza cortante es calculada de manera similar al caso de vigas. La única diferencia está en el peralte efectivo d que, para el caso los muros se toma igual a $0.8l_w$. Para una longitud de muro dada, el peralte efectivo dependerá de la cuantía y de la distribución del acero vertical. Sin embargo, se puede demostrar que la hipótesis convencional de tomar $d=0.8\ l_w$ es razonable.

Resultados experimentales han indicado que, manteniendo las otras variables iguales, se mejora la respuesta histerética de muros cuando el refuerzo en el alma es mediante varillas de diámetro pequeño colocadas a separaciones pequeñas.

Con objeto de garantizar la resistencia del muro al agrietamiento diagonal del concreto, es necesario colocar una cuantía mínima de refuerzo horizontal. Para valores normales de resistencia a la compresión del concreto y varillas Grado 400 (Grado 400 se refiere a $f_y = 400$ MPa $\approx 4{,}200$ kg/cm^2), la cuantía mínima es igual a 0.25%. Esta cantidad de refuerzo es adecuada para controlar los cambios volumétricos del concreto.

De manera similar al caso de vigas y columnas, la resistencia al cortante disminuye en regiones donde fluye el refuerzo a flexión. Por tanto, es importante diseñar y detallar refuerzo horizontal por corte adicional para la zona de la articulación plástica.

El deslizamiento por cortante en muros estructurales esbeltos es menor crítico que en vigas debido a la carga axial actuante y a la distribución uniforme del refuerzo vertical. Este último ayuda a controlar el agrietamiento horizontal y resiste el cortante mediante la acción de dovela (transversal al eje de la varilla) y cortante-fricción. En planos de deslizamiento potencial es recomendable colocar el acero vertical a una separación igual al espesor del muro. Estudios experimentales han demostrado que la falla por deslizamiento puede retrasarse si el esfuerzo cortante nominal es menor de $3\sqrt{f'_c}$, en kg y cm^2.

3.4 Confinamiento e inestabilidad

Como se estudió en el capítulo sobre confinamiento, un adecuado confinamiento del concreto incrementa su resistencia a la compresión y su capacidad de deformación (ductilidad). Cuando fluye el refuerzo a flexión del muro, los esfuerzos a compresión en el concreto aumentan para equilibrar la tensión, pero si el concreto no está confinado, puede alcanzar la falla rápidamente. En este caso la falla se caracterizaría por el aplastamiento y desconchamiento del concreto en una gran porción de los extremos del muro. El confinamiento debe extenderse sobre la zona de la articulación plástica.

Para evitar una posible falla por inestabilidad de la zona a compresión del muro (Fig. 11) es recomendable que el espesor del muro sea mayor o igual a un décimo de la altura de la planta baja del edificio. El pandeo del refuerzo principal a compresión se puede retrasar si éste se confina con estribos cerrados separados a seis veces el diámetro máximo nominal de la varilla vertical del muro.

Aun cuando el muro se confine, es probable que pueda fallar por inestabilidad lateral del núcleo confinado. Esta falla puede evitarse si se colocan patines en los extremos del muro. En la Fig. 12 se muestran detalles típicos del refuerzo transversal en los patines.

3.5 Diseño

Las NTC-Concreto contienen requisitos para el diseño y detallado de muros estructurales sujetos a fuerzas horizontales en su plano. No se pretende en este acápite transcribir dichos requerimientos; solamente se comentarán algunos de ellos.

Los edificios, en los cuales los muros resistan la totalidad de las fuerzas laterales, se diseñan con un factor de comportamiento sísmico Q=3 (Art. 4.5.2) si se satisfacen los requerimientos para elementos extremos; de otra manera se emplea Q=2. El valor Q=3 presupone que la capacidad de disipación de energía y la ductilidad del muro estructural son buenas, de aquí que es indispensable una inspección y supervisión estrictas durante la construcción. De particular relevancia es la colocación del refuerzo transversal, traslapes y anclaje según los planos estructurales. En el Art. 4.5.2.a se indica la distribución del refuerzo vertical por flexión en la longitud del muro y el corte del refuerzo. La razón de colocar el refuerzo vertical distribuido en muros robustos obedece a consideraciones de resistencia al cortante. Para muros con relación de aspecto h_w / l_w mayor que 1.2, el corte del refuerzo longitudinal se hará a una altura igual a $1.2 l_w$. La razón es la de asegurar un anclaje adecuado del refuerzo en la zona de la articulación plástica.

Para usar Q=3, los elementos extremos de los muros deben confinarse con estribos colocados a pequeñas separaciones. Los estribos deberán ser cerrados y de una pieza, ya sea sencillos o sobrepuestos. El diámetro menor será varilla del No. 10 (se refiere a 10 mm, es decir, de 3/8 pulg). Los estribos deben rematar en una esquina con dobleces de 135 °, seguidos de tramos rectos de no menos de 10 diámetros de largo. La separación máxima no debe exceder de 10 cm. El cumplimiento de los requisitos de detallado anteriores es esencial para confinar adecuadamente el concreto, evitar pandeo del refuerzo longitudinal y mejorar la capacidad de disipación de energía y ductilidad del muro.

La expresión para calcular la cuantía del refuerzo vertical p_v (Art. 4.5.2c, Ec. 4.7) indica que si h_w/l_w disminuye, la cantidad de acero vertical aumenta, lo cual es consistente con lo discutido para muros estructurales robustos (ver sección 4).

$$p_v = 0.0025 + 0.5\ (2.5 - \frac{h_w}{l_w})\ (p_h - 0.0025)$$

Las NTC-Concreto indican que la separación máxima del refuerzo será de 35 cm. Esta recomendación está basada en criterio y en práctica tradicional, y no en estudios específicos.

El esfuerzo cortante máximo se limita a $2\sqrt{f^*_c}$, en kg y cm^2, para evitar el aplastamiento del concreto asociado a fallas por compresión diagonal. La sección crítica por corte está a media altura de entrepiso.

4. MUROS ESTRUCTURALES ROBUSTOS

4.1 Tipos de Muros

Se denomina muro estructural robusto a aquél con una relación de aspecto h_w/l_w menor o igual que dos. De acuerdo a su comportamiento se les puede clasificar en tres categorías:

1. Muros elásticos. Es usual que la resistencia de muros bajos sea tan alta que respondan en el intervalo elástico ante sismos intensos. La mayoría de los muros pertenece a este tipo.
2. Muros que pueden cabecear. Es el caso de muros que resisten la mayor parte de la carga lateral aunque soportan una carga vertical relativamente baja. En este caso la capacidad del muro está limitada por la resistencia a volteo. Si la cimentación se diseña para este tipo de comportamiento el muro permanece elástico.
3. Muros dúctiles. En algunas ocasiones no es posible diseñar la cimentación de manera que los muros permanezcan en el intervalo elástico. Entonces es necesario diseñar los muros para que exhiban un comportamiento inelástico limitado.

Es común que la resistencia a flexión de estos muros sea tan alta que es difícil desarrollarla sin que fallen por corte antes. Es importante notar que este tipo de falla puede aceptarse si las demandas de ductilidad (desplazamiento) son mucho menores que las requeridas para muros esbeltos o acoplados. Estos muros deben identificarse como muros con ductilidad restringida.

4.2 Resistencia a la Flexión

Para resistir el momento flexionante, usualmente es suficiente colocar refuerzo vertical mínimo distribuido uniformemente. El principal problema es cómo resistir la fuerza cortante. Al igual que para los muros esbeltos, la distribución uniforme del acero vertical ayuda a resistir el deslizamiento por cortante mediante los mecanismos de cortante-fricción y acción de dovela de las varillas.

4.3 Resistencia al Cortante

En los primeros ensayes ante carga lateral realizados en muros bajos, se aplicó la fuerza concentrada en las esquinas de los tableros. Los muros robustos, cargados de esta manera, pueden resistir cargas importantes debido a la formación de un puntal de compresión interno. Sin embargo, los muros robustos son generalmente cargados mediante cargas puntuales transmitidas por los diafragmas de piso en cada nivel. En estos casos el mecanismo resistente de puntales de compresión no es tan eficiente como en el caso de carga concentrada.

Al igual que en los muros estructurales esbeltos, es indispensable la colocación de refuerzo horizontal para resistir parte del cortante. Sin embargo, también es necesario colocar refuerzo vertical para tomar el cortante. Si observamos la Fig. 13, es claro que para equilibrar el componente vertical del puntal a compresión, es necesario un tensor, es decir, refuerzo vertical. Se concluye que el cortante solamente se puede resistir si se coloca refuerzo vertical. La cuantía mínima de refuerzo, tanto horizontal como vertical, será igual a 0.25% como para el caso de muros esbeltos.

En la Fig. 14 se presentan esquemáticamente los modos de falla por cortante de muros robustos. Se produce una falla por tensión diagonal (Fig. 14a) cuando el refuerzo horizontal es insuficiente para controlar la grieta. La resistencia a tensión diagonal depende de cómo se aplica la fuerza cortante. Así, si se puede distribuir la fuerza a lo largo del muro, el agrietamiento por tensión diagonal no será sinónimo de falla (Fig. 14b).

Si el esfuerzo cortante es elevado y el refuerzo horizontal es adecuado, el concreto puede aplastarse bajo la compresión diagonal (Fig. 14c). Este caso es típico en muros con patines con una resistencia a la flexión elevada. A menudo, el aplastamiento puede extenderse sobre la longitud del muro (Fig. 14d). La falla por compresión diagonal conduce a una rápida pérdida de resistencia y debe evitarse cuando se diseñen los muros. Los reglamentos de construcción (ver sección 3.5) limitan el esfuerzo cortante máximo que se puede aplicar para asegurar que la falla por compresión no disminuya la ductilidad disponible.

Como se mencionó arriba, las fallas por compresión o tensión diagonales se evitan si se limita el esfuerzo cortante nominal y si se coloca refuerzo horizontal. Por tanto las deformaciones inelásticas (fluencia) ocurrirán en el refuerzo vertical. Después de alguno ciclos de carga, es posible que ocurra un deslizamiento de la base. Este fenómeno reduce la resistencia y la rigidez, la última particularmente a bajos niveles de desplazamiento, lo que trae como consecuencia una disminución en la energía disipada. Debido a este desplazamiento, la fuerza de compresión en la zona a compresión de la flexión, se transmite a través de superficies no uniformes de la grieta. Esto conduce a un mayor deterioro que se manifiesta en aplastamiento y desprendimiento del concreto. El daño en el concreto, a su vez, reduce la adherencia del acero vertical y la rigidez de la acción de dovela. Eventualmente el principal mecanismo resistente será el pliegue del refuerzo vertical.

4.4 Control del Deslizamiento por Cortante

Ensayes en muros han indicado los efectos negativos que desplazamientos por corte excesivos producen en la respuesta histerética. También han evidenciado el mejoramiento del comportamiento cuando se coloca refuerzo diagonal que cruza el plano de deslizamiento para reducirlo y para resistir el cortante de deslizamiento. En las Figs. 15a y 15b se presentan las respuestas histeréticas de un muro robusto que falló por deslizamiento sobre la base. La respuesta de la Fig. 15c corresponde a un muro con refuerzo diagonal (Fig. 16) diseñado para resistir el 30% del cortante de deslizamiento; es notable el cambio en las curvas. Para controlar el desplazamiento en la base se ha propuesto que el 50% del cortante sea resistido por acero diagonal y el resto por acción de dovela. Para este último se ha propuesto que sea igual a 0.25 veces la resistencia a tensión del refuerzo vertical.

4.5 Control de la Tensión Diagonal

Para resistir la fuerza de tensión diagonal se debe colocar refuerzo horizontal que equilibre el cortante que actúa sobre un plano de falla supuesto con una inclinación a 45°. Si existe acero diagonal (ver sección anterior) se deberá considerar el componente horizontal de la resistencia.

4.6 Diseño

Los comentarios de diseño según NTC-Concreto se presentan en la sección 3.5.

5. SISTEMAS MIXTOS MURO - MARCO

Es común el empleo de muros estructurales esbeltos en combinación con marcos de acero o de concreto reforzado. En estos casos, los muros se construyen entre columnas, tal que los elementos extremos del muro sean las propias columnas. El sistema mixto marco-muro combina las ventajas de ambos componentes. Así, marcos dúctiles pueden disipar energía en los pisos superiores de un edificio. Por otro lado, dada la rigidez de los muros, las distorsiones de entrepiso (desplazamiento relativo entre altura) estarán dentro de los límites permisibles.

Ante cargas laterales, un marco se deforma principalmente en modo de corte (Fig. 17), mientras que un muro se comporta como un voladizo vertical dominado por flexión. Dada la compatibilidad de desplazamientos obligada por las losas de piso, el marco y los muros comparten la resistencia en los pisos inferiores pero se oponen en los superiores.

En comparación con un muro aislado ante cargas laterales, la interacción con el marco produce menores momentos máximos (en la base), pero fuerzas cortantes mayores. Esto aumenta la tendencia a una falla por corte. Lo anterior es particularmente importante si estudiamos la vieja práctica de algunos despachos de cálculo estructural de diseñar el marco (sin muros) para resistir la carga gravitacional y el (los) muro(s) de manera separada (sin marco) para resistir la carga lateral total. Puesto que para un muro conectado a un marco, el momento máximo es más bajo que el obtenido del análisis de un muro como voladizo, el diseño por flexión sería conservador. Sin embargo, el diseño por corte sería peligrosamente no conservador ya que los cortantes en el muro diseñado como voladizo son menores que los obtenidos en muros conectados a marcos.

Mientras más flexibles son los muros, mayores serán los cortantes que deben ser resistidos por las columnas de los marcos. En realidad la contribución de los muros a tomar cortante es en los pisos inferiores.

En algunas ocasiones, la resistencia y rigidez de la cimentación no son suficientes para evitar el levantamiento del muro por cabeceo. Este fenómeno se traduce en cargas axiales mayores sobre el muro que aumentan su resistencia a la flexión. Este aumento, extrañamente quizá, no es conveniente, ya que aumenta la fuerza cortante. Si este incremento no es tomado en cuenta se puede dañar al muro por corte prematuramente. Además, el levantamiento del muro introduce cortantes en vigas transversales para los que generalmente no son diseñadas. Estas fuerzas cortantes se traducen en fuerzas axiales a tensión en columnas en el extremo opuesto de las vigas. Si esta fuerza de tensión no se consideró en el diseño y detallado de las columnas, es posible que se formen articulaciones plásticas en zonas no detalladas para ello.

Análisis dinámicos más refinados han indicado un buen comportamiento de sistemas mixtos bien detallados en los cuales los muros se extienden de la base a parte de la altura del edificio.

Los comentarios hechos en las secciones anteriores sobre el confinamiento, anclaje y deslizamiento son aplicables a este caso.

6. MUROS ESTRUCTURALES ACOPLADOS

6.1 Ventajas de los Muros Acoplados

Una desventaja potencial de los muros estructurales con comportamiento controlado por flexión es que la mayor parte de la disipación de energía ocurrirá mediante plastificación del refuerzo a flexión, lo que está asociado al peligro de una falla por deslizamiento en la articulación plástica. Este tipo de daño es difícil de reparar puesto que, por lo general, los muros resisten la mayor parte de las cargas gravitacionales del edificio.

Si consideramos el caso de dos muros acoplados, la rigidez del sistema aumentará con el peralte de las vigas de acoplamiento. Sin embargo, la principal ventaja de este tipo de sistema está en su comportamiento inelástico. La deformación de los muros ante cargas laterales causan grandes desplazamientos relativos entre los extremos de las vigas de acoplamiento (Fig. 18). Esto provoca la formación de articulaciones en los extremos mucho antes de la formación de las articulaciones en los muros mismos. La estructura puede disipar una cantidad significativa de energía a través de la sola fluencia de las vigas acopladas. Debido a la respuesta del edificio en el segundo y tercer modo de vibración, aun en medio ciclo de desplazamiento del muro, las vigas de acoplamiento son sometidas a varias ciclos de momento.

Una ventaja adicional del sistema es que si las vigas son severamente dañadas durante un sismo, se pueden reparar de manera relativamente fácil sin dejar al edificio fuera de servicio. Aun más, si las vigas son destruidas completamente, el edificio tiene la redundancia estructural que le brindan los muros trabajando de manera independiente, lo que evita su colapso.

6.2 Criterio de Diseño

Para garantizar un comporamiento adecuado de los muros acoplados se debe satisfacer que:

1. La formación de articulaciones plásticas en las vigas de acoplamiento debe ocurrir antes que la plastificación de los muros; y
2. Las vigas de acoplamiento deben ser detalladas para obtener buenas características de disipación de energía.

El primer requisito es satisfecho si se diseñan los muros de manera que la resistencia nominal al cortante sea mayor que el cortante consistente cuando se alcanza la capacidad a flexión del muro. Esta capacidad se calcula considerando la reducción en la carga axial debido a la formación de articulaciones plásticas en las vigas de acoplamiento. En efecto, al fluir las vigas, las fuerzas cortantes en ellas se traducen en una reducción en las fuerzas axiales en los muros. Si la carga axial neta en el muro de sotavento es baja, se reduce la resistencia al corte y se favorece la degradación por deslizamiento.

Respecto a la resistencia de las vigas de acoplamiento, es importante señalar que la relación claro-peralte de las vigas de acoplamiento es menor de dos, lo que resulta en elementos vulnerables a fallas por cortante.

6.3 Diseño de Vigas de Acoplamiento

Las primeras vigas de acoplamiento se reforzaron por corte de manera convencional, es decir, aplicando conceptos para vigas esbeltas y colocando estribos ortogonales al eje a baja separación. Sin embargo, su respuesta ante sismos ha sido deficiente. Las vigas así reforzadas fallan por tensión diagonal con degradación muy severa o por deslizamiento cerca del muro (Figs. 19a y 19b). Las razones de este comportamiento son los altos esfuerzos cortantes nominales que aceleran la degradación por corte y la distribución no lineal de esfuerzos, la cual es diferente de la supuesta por la teoría convencional de vigas. En efecto, el refuerzo longitudinal de la viga permanece a tensión en todo el claro, de manera que el cortante se transmite por medio de un puntal diagonal (Fig. 19c).

Puesto que el concreto se degradará ante ciclos de carga, es necesario resistir la compresión diagonal a través de varillas diagonales que puedan resistir todo el componente inclinado de la fuerza cortante. El mínimo número de varillas es cuatro. Se deberán colocar estribos cerrados a 10 cm máximo para evitar el pandeo de dicho refuerzo. El refuerzo deberá anclarse en el muro para permitir su fluencia. Según NTC-Concreto el anclaje será igual a 1.5 veces la longitud de desarrollo de las varillas. Este incremento pretende disminuir la concentración de esfuerzos en el anclaje. En la Fig. 20 se presenta el comportamiento de vigas de acoplamiento reforzadas convencionalmente y reforzadas con acero diagonal. Es importante observar las excelentes características de disipación de energía de estas últimas. Los detalles del refuerzo de una viga de acoplamiento se ilustran en la Fig. 21. Para evitar el desprendimiento del concreto agrietado, es necesario colocar refuerzo horizontal y vertical mínimo que funcionen como una canasta. Este refuerzo debe cumplir los requisitos para acero por cambio volumétricos y se colocará en dos capas, próximas a las caras de la viga, por afuera del refuerzo diagonal.

7. MUROS DIAFRAGMA DE CONCRETO REFORZADO

7.1 Características

El comportamiento sísmico de marcos con muros diafragma (o de relleno) de concreto reforzado depende del espesor relativo de los últimos con respecto a las dimensiones de vigas y columnas del marco. En efecto, si los muros diafragma son muy delgados, el marco se deformará como un marco sin muros; en este caso la energía se disipará en las vigas y columnas. Por el contrario, si el muro tiene un espesor alto, el marco con muros responderá como un muro estructural, de manera que la energía se disipará mediante fluencia en la base de la estructura. Para muros diafragma de espesor intermedio, el marco con muros se comportará como un muro estructural para bajos niveles de desplazamiento. Para desplazamientos elevados, los muros diafragma se comportarán como puntales equivalentes de compresión y la estructura responderá como un marco arriostrado. Debido a la degradación gradual de los tableros se logrará una significativa cantidad de energía disipada. Aunque el comportamiento de este tipo de sistemas no es tan bueno como el de muros acoplados, ofrece un incremento en resistencia, rigidez y disipación de energía comparado con un marco simple, siempre y cuando el marco y los muros diafragma se diseñen y detallen adecuadamente.

7.2 Criterios de Diseño

Los posibles problemas de diseño en el empleo de muros diafragma y sus soluciones son:

1. Flexibilidad de los tableros. Puesto que los muros diafragma son mucho más rígidos que el marco, es posible que la estructura falle por fluencia en la base. Para este caso, se recomienda que los muros diafragma se compongan de tableros separados por juntas verticales, o bien que la carga asociada a la falla por flexión sea superior que la que produciría el aplastamiento del puntal de compresión.
2. El puntal diagonal de compresión introduce en las columnas fuerzas cortantes elevadas. Para evitar

una falla en la columnas se requiere usar una alta cantidad de refuerzo transversal de manera que resista todo el cortante transmitido cuando el muro diafragma se agriete. Así, la resistencia al corte en los extremos de la columna deberá ser mayor que la carga de agrietamiento del muro diafragma (por lo general el esfuerzo de agrietamiento es del orden de $1.8\sqrt{f'_c}$, en kg y cm^2).

3. El muro diafragma puede desprenderse del marco y no disipar energía. Para evitar ello se recomienda la colocación de una cuantía mínima de acero vertical y horizontal igual a 0.0025 con una separación máxima de varillas de 30 cm. Este refuerzo deberá estar anclado al marco.

Lo discutido anteriormente es válido para el caso que se quiera que el muro diafragma contribuya a la resistencia y rigidez ante cargas laterales del edificio. Si el muro es divisorio únicamente, se deberá separar del marco por medio de una junta elástica.

8. DETALLADO

En las secciones anteriores se han hecho varias observaciones respecto a la influencia del detallado en el comportamiento de los muros estructurales. A continuación se enfatizan los aspectos de juntas de construcción y anclaje. La importancia del confinamiento ha sido destacada en otras secciones.

8.1 Juntas de Construcción

Los muros estructurales de concreto normalmente se construyen colando por tramos, mismos que quedan separados por juntas de construcción. Estas juntas tienen, a menudo, una resistencia dudosa. En efecto, durante la compactación del concreto el material más pesado, los agregados, se precipitan al fondo de la capa de colado. Por tanto, en la parte superior existirá un mayor contenido de pasta con relación agua/cemento más alta (de acuerdo a Abrams, a mayor relación agua/cemento, menor es la resistencia).

Para evitar el deslizamiento a lo largo de juntas horizontales es necesario colocar suficiente refuerzo vertical (acero en el alma) con baja separación o acero diagonal (ver sección 4.4) para resistir el cortante mediante el mecanismo de fricción-cortante. Debido al desplazamiento relativo a lo largo de la junta rugosa, la junta se abre (un valor típico es del orden de 0.2 mm). Si algunas varillas cruzan la junta, éstas quedarán sometidas a fuerzas de tensión que serán equilibradas por la compresión a ambos lados de la junta. La resistencia asociada a este mecanismo es proporcional al área transversal del acero que atraviesa la junta y al esfuerzo en las varillas. Para desplazamientos relativos pequeños (del orden de 0.2 mm o menos), el mecanismo de cortante-fricción es razonable. Por tanto, el cortante rasante resistente será función de la fuerza normal a la junta y de la fuerza desarrollada por las varillas que la cruzan multiplicados por un coeficiente de fricción. De esto último se desprende la necesidad de incrementar la rugosidad de juntas de construcción. Las juntas de construcción deben estar libres de polvo, partículas o cualquier otra sustancia que afecte la adherencia con el nuevo concreto. Ensayes de laboratorio han indicado que el uso de aditivos no modifica sustancialmente la resistencia al corte.

El refuerzo vertical mínimo es suficiente para controlar el desplazamiento en la base si el esfuerzo axial sobre el muro es igual o mayor de 4.2 kg/cm^2.

8.2 Anclaje

El refuerzo vertical en muros, ya sean esbeltos o robustos, debe anclarse en la base del muro; esto es evidente. Parecería que el anclaje del refuerzo del alma no es tan necesario en la parte superior del muro; sin embargo, es de similar importancia, en particular en muros bajos en los cuales la mayor parte del cortante es resistido por el refuerzo vertical después del agrietamiento del concreto. Se deberán emplear ganchos en la parte superior para garantizar un adecuado anclaje. Un aspecto relevante son las uniones de varillas, en particular cerca de la base del muro. Las NTC-Concreto prohiben traslapes de cualquier tipo en la zona de la articulación plástica. En el caso de muros, la colocación de traslapes en zonas con esfuerzos altos es más detrimental que en el caso de columnas o vigas debido a la falta de un adecuado confinamiento lateral (hacia fuera del plano del muro) por la geometría del elemento.

El refuerzo horizontal se debera anclar en los extremos y, de preferencia, dentro de los elementos extremos confinados.

En ocasiones no se presta suficiente atención a la separación entre las varillas, particularmente aquellas colocadas en los elementos extremos. Es común observar el uso de paquetes de varillas de gran diámetro muy próximos entre sí lo que dificulta la adecuada colocación y compactación del concreto. Una mala práctica de colado se traduce en hoquedades que reducen la adherencia del refuerzo, lo que a su vez conduce a una disminución en la resistencia, rigidez y capacidad de desplazamiento del muro. Se debe tener especial cuidado en supervisar dicha condición.

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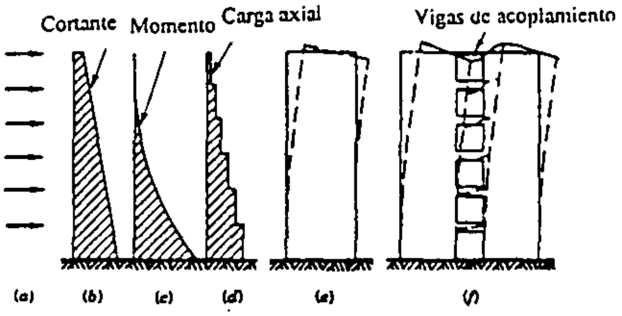


Fig. 1 Variación de la fuerza cortante, momento y carga axial en un muro estructural aislado

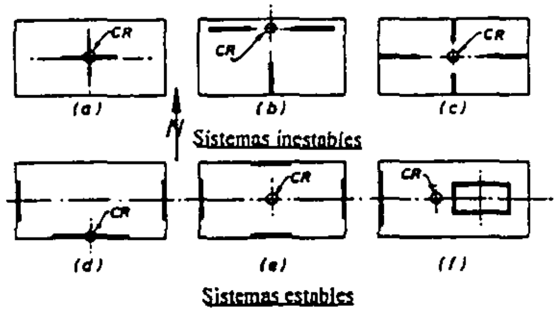


Fig. 2 Ejemplos de estabilidad torsional en sistemas de muros estructurales

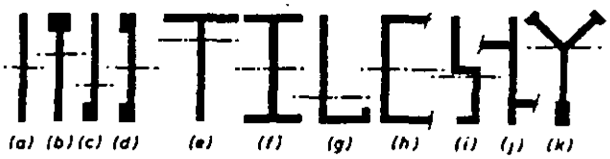


Fig. 3 Secciones transversales comunes de muros estructurales

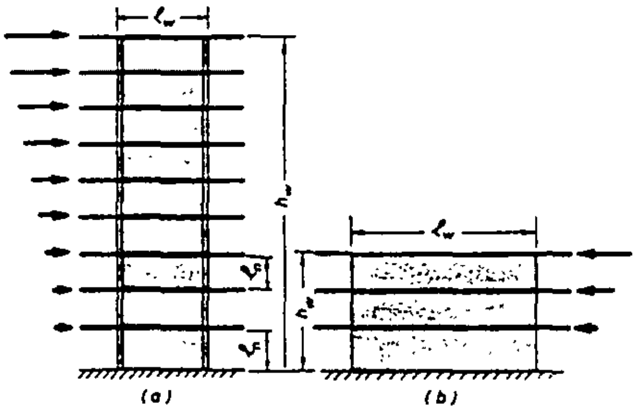


Fig.4 Muros estructurales esbeltos y robustos

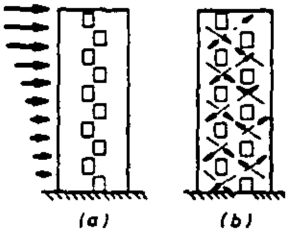


Fig. 5 Resistencia al corte afectada por aberturas en muros

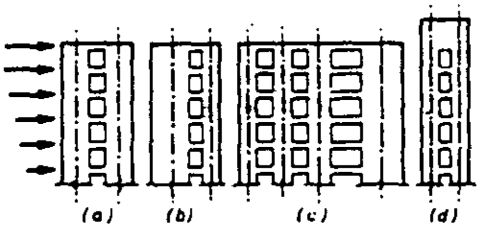


Fig. 6 Tipos de muros estructurales acoplados

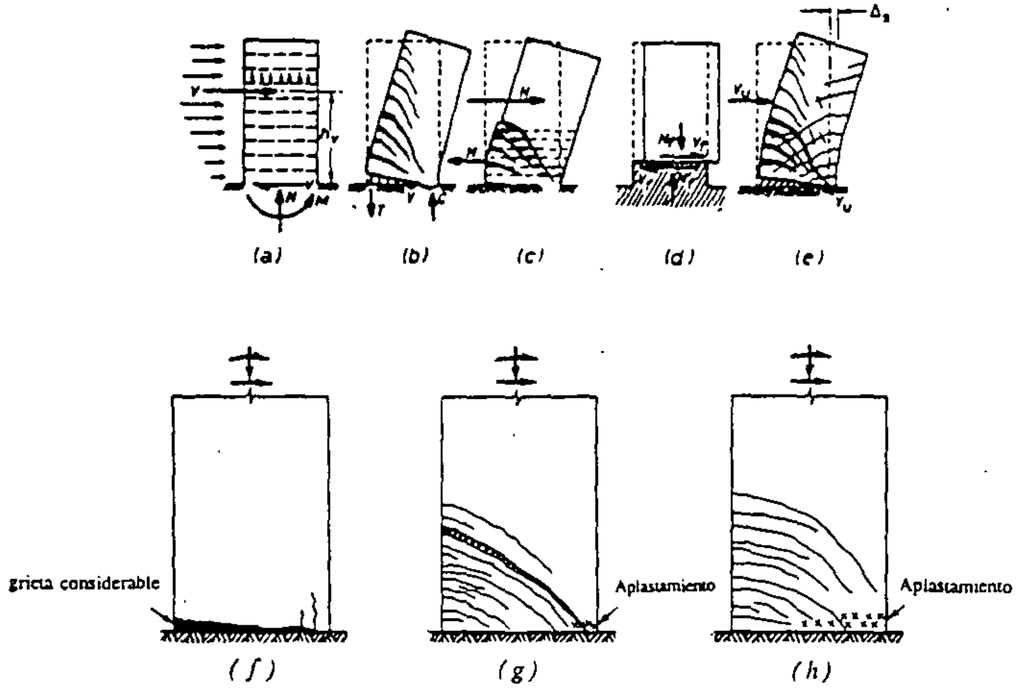


Fig. 7 Modos de falla en muros esbeltos

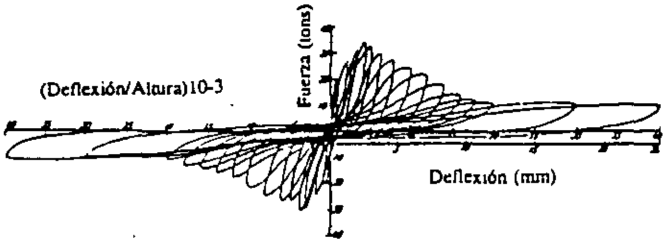


Fig. 8 Respuesta histerética dominada por la resistencia al corte de un muro estructural

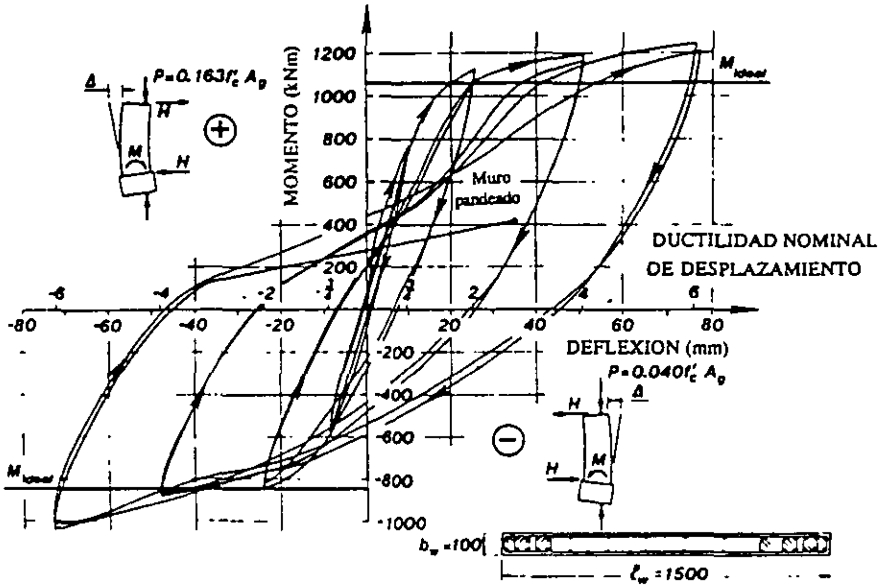


Fig. 9 Respuesta histerética estable de un muro estructural dúctil

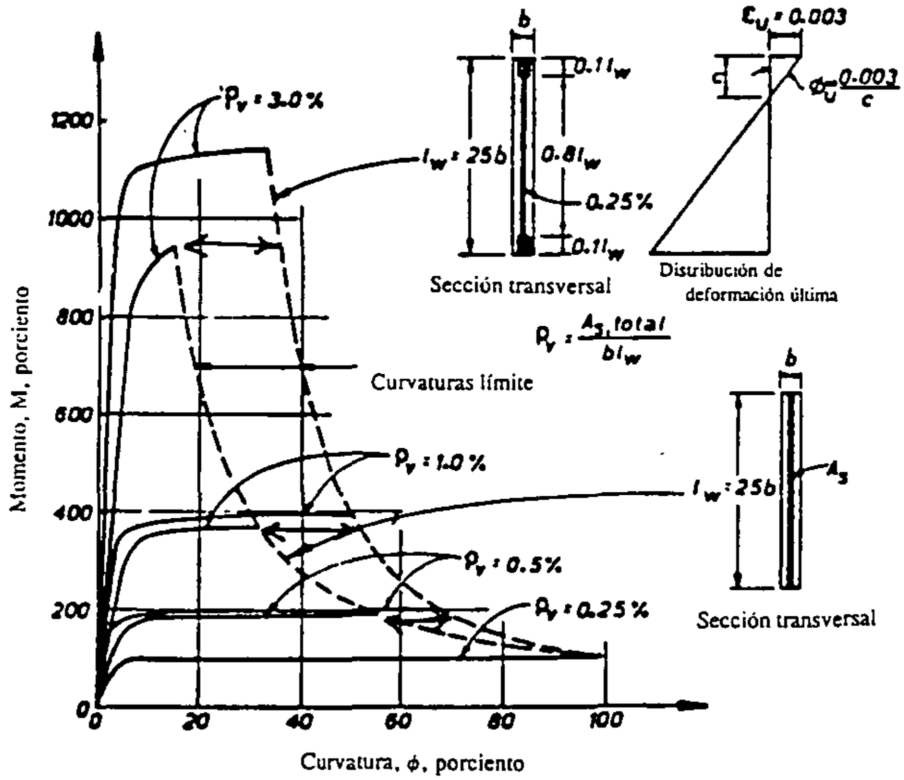
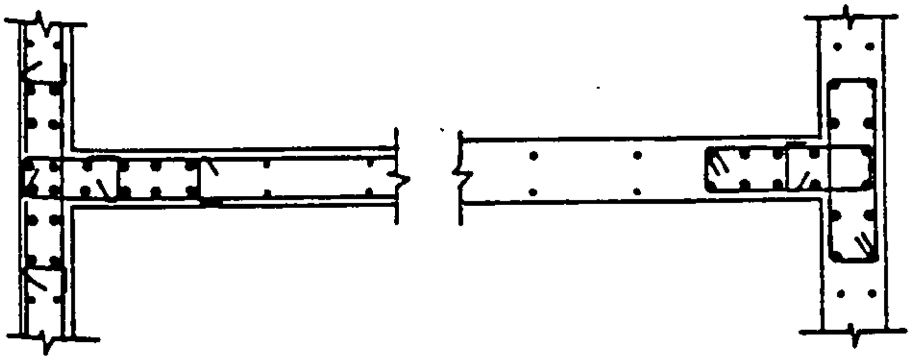


Fig. 10 Efecto de la distribución del refuerzo y de la cuantía en la resistencia a flexión y en la curvatura

Fig. 11 Pandeo en la región de la articulación plástica de un muro estructural



Sección transversal
a nivel del refuerzo
horizontal

Sección transversal
entre niveles de refuerzo
horizontal

Fig. 12 Detalles de confinamiento en patines de muros

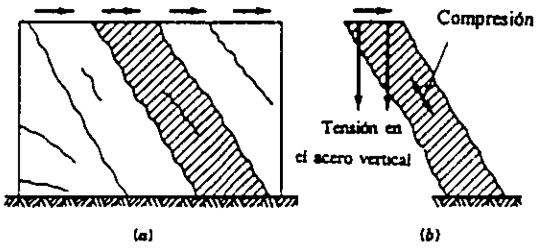


Fig. 13 Puntal de compresión entre grietas para muros robustos

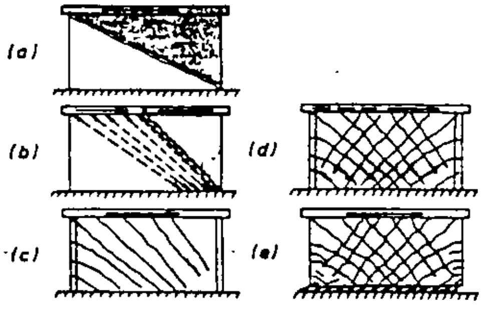


Fig. 14 Modos de falla de muros robustos

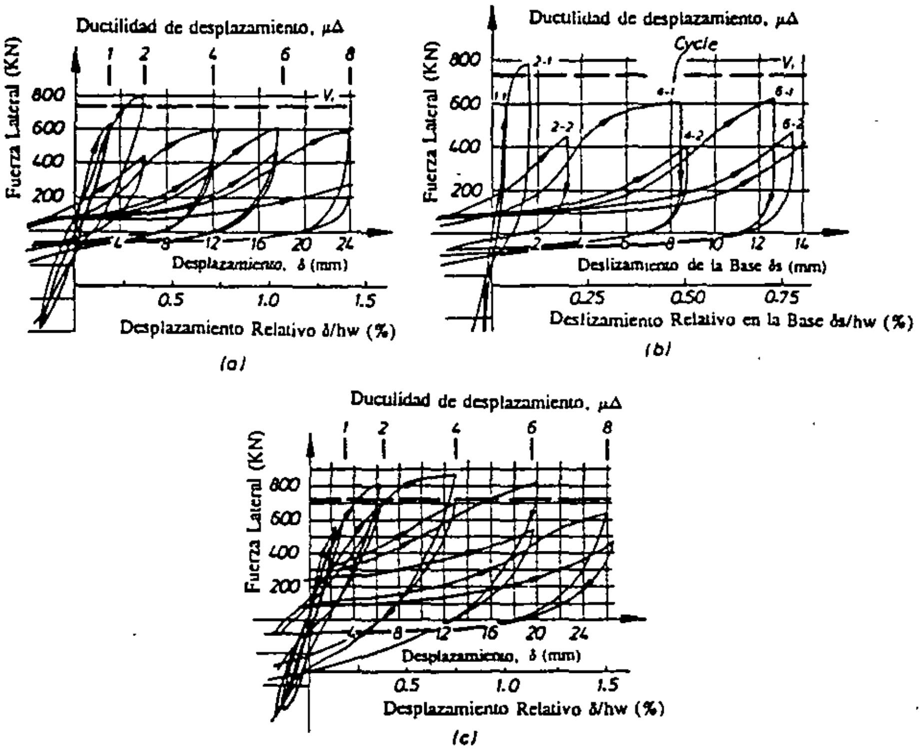


Fig. 15 Respuesta histerética de muros robustos con patines con falla controlada por deslizamiento en la base

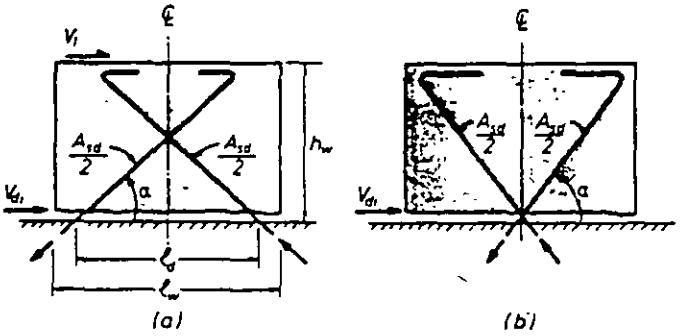


Fig. 16 Refuerzo diagonal en muros robustos

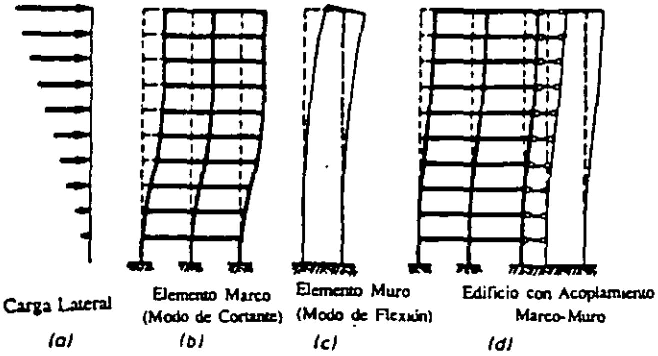


Fig. 17 Patrones de deformación ante cargas laterales de un marco, un muro y un sistema mixto muro-marco

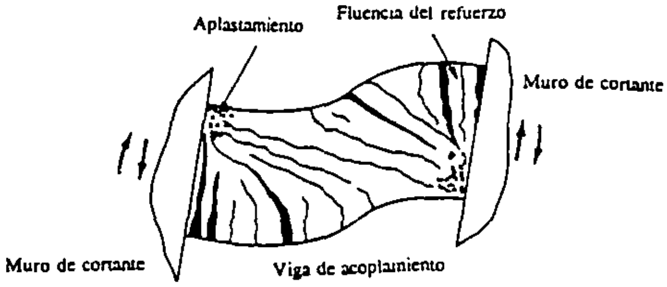


Fig. 18 Deformación y agrietamiento de vigas de acoplamiento

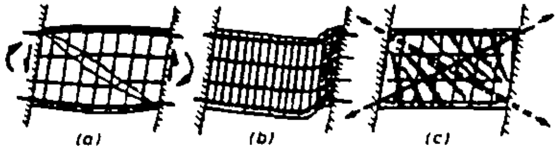


Fig. 19 Mecanismos de resistencia al corte de una viga de acoplamiento

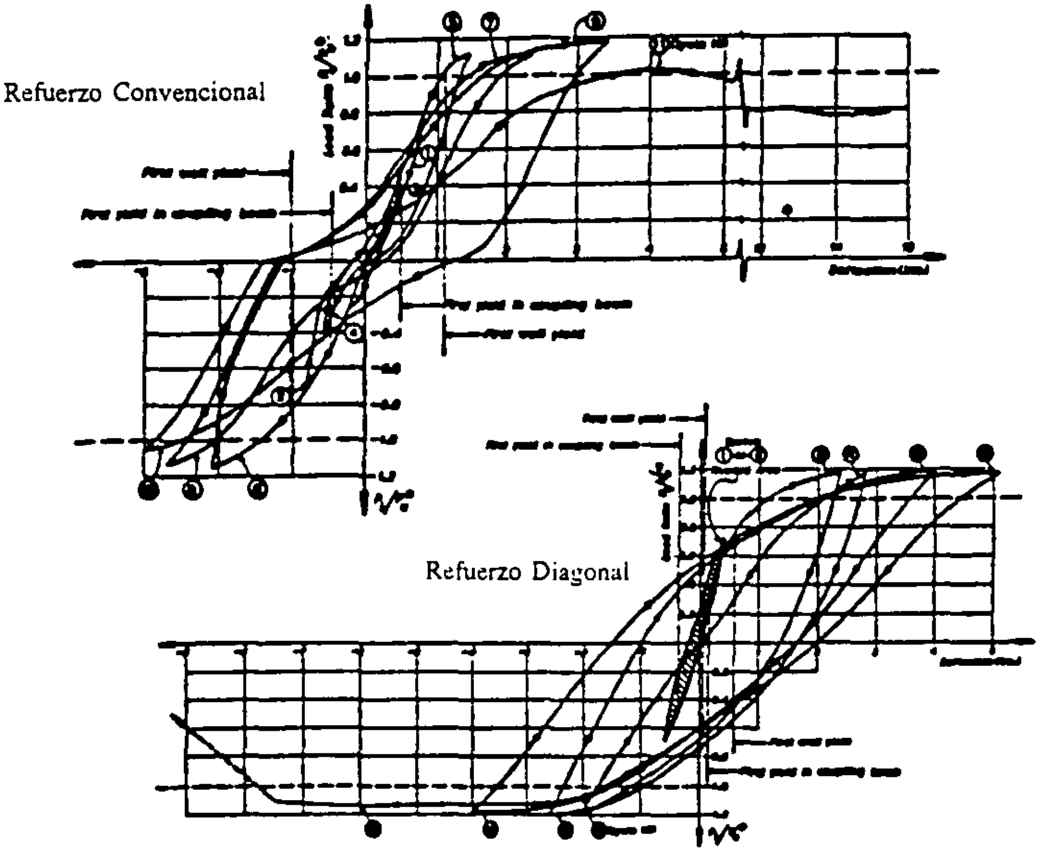


Fig. 20 Comportamiento de vigas de acoplamiento reforzadas convencionalmente y con acero diagonal

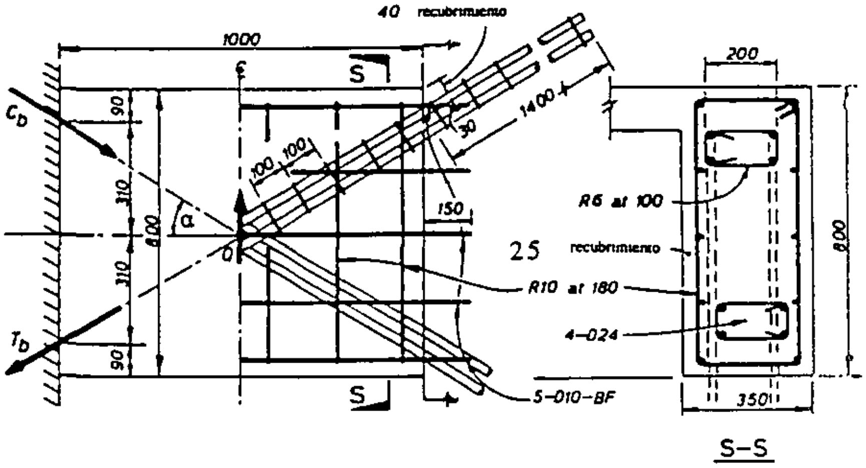


Fig. 21 Detalles del refuerzo de una viga de acoplamiento

COMPORTAMIENTO Y DISEÑO DE ESTRUCTURAS DE CONCRETO REFORZADO

UNIONES DE ELEMENTOS

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1. INTRODUCCION

El diseño de uniones ha sido un aspecto que no ha recibido la debida atención por parte de investigadores, y de profesionales de la construcción y diseño. A menudo se argumenta que la importancia que se la ha dado recientemente a las uniones, en particular la de vigas con columnas, es exagerada ya que no existe evidencia abundante de fallas en sismos pasados. Esta idea se basa en que los problemas en marcos de concreto reforzado se han presentado por diseños mal concebidos en vigas y, particularmente, por un detallado inadecuado de columnas. Sin embargo, sismos recientes, como el de El Asnam en 1980 (Fig. 1), los de México en 1985, San Salvador en 1986 , Loma Prieta en 1989, y el de Los Angeles en 1994, han evidenciado fallas por corte y de anclaje en uniones viga-columna.

Es común que los diseñadores olviden el detallado de las uniones. Se deja al constructor la definición de detalles críticos que influyen en el comportamiento de la estructura. Las uniones son críticas porque aseguran la continuidad del edificio y porque transmiten fuerzas de un elemento a otro. Así, las cargas y fuerzas deben transmitirse del sistema de piso a las trabes, de ellas a las columnas, y de las últimas a la cimentación. La transferencia de fuerzas entre los elementos depende del detallado cuidadoso de las uniones y de la supervisión minuciosa que asegure que la fabricación y la construcción sigan las instrucciones o intenciones del diseñador.

2. CRITERIOS DE DISEÑO DE UNIONES VIGA-COLUMNA

Los criterios de diseño de uniones viga-columna se pueden formular como sigue:

1. La resistencia de la unión debe ser mayor o igual que la máxima demanda que corresponda a la formación del mecanismo de colapso del marco. Esto eliminará la necesidad de reparar una región inaccesible y que sufre deterioros de resistencia y rigidez considerables si se somete a acciones cíclicas en el intervalo inelástico.
2. La resistencia de la columna no debe afectarse por una posible degradación de resistencia de la unión.
3. Ante sismos moderados, las uniones deben responder en el intervalo elástico.
4. Las deformaciones de la unión no deben contribuir significativamente al desplazamiento de entrepiso.
5. El refuerzo en la unión, necesario para garantizar un comportamiento satisfactorio, no debe dificultar la construcción. Una unión típica conecta elementos provenientes de tres direcciones; se debe evitar la interferencia de las varillas que vienen de todas las direcciones.

3. COMPORTAMIENTO ESPERADO

Puesto que la respuesta de uniones viga-columna está controlada por mecanismos de corte y adherencia, que tiene un comportamiento histerético pobre, no es posible considerar a la unión como una fuente importante de disipación de energía. Por tanto, la unión debe experimentar bajos niveles de agrietamiento y plastificación. La unión debe detallarse de manera que sus deformaciones no contribuyan significativamente a la distorsión del entrepiso (se entiende por distorsión al cociente del desplazamiento relativo entre la altura de entrepiso). Uniones bien diseñadas contribuyen en 20% a la distorsión total.

Como ejemplo, una unión de fachada estará sometida a las fuerzas indicadas en la Fig. 2. En la Fig. 2b se presenta la distorsión angular de la unión. El agrietamiento de las vigas en las caras de las columnas, y el fisuramiento de las columnas en las partes superior e inferior de las vigas son el resultado del deslizamiento del refuerzo a través de la unión. Es común suponer en el análisis de edificios que las condiciones de apoyo de las vigas en las columnas son iguales a un empotramiento. En realidad, el refuerzo de las vigas se deslizará aun para bajos niveles de esfuerzo, de manera que un empotramiento perfecto no es posible. La unión se deforma en cortante por las fuerzas resultantes que obran en la unión (Fig. 2c), las cuales producen tensión a lo largo de una diagonal de la unión y compresión a lo largo de la otra. Las primeras grietas diagonales aparecen cuando los esfuerzos principales de tensión exceden la resistencia a tensión del concreto. Puesto que las grietas son similares a las grietas por cortante en una viga, las primeras recomendaciones de diseño se basaron en ecuaciones adaptadas de requerimientos de corte para vigas. Es importante notar que las magnitudes de las fuerzas a las que se somete una unión son varias veces las aplicadas en vigas y columnas.

Ante sismos, las vigas que llegan a la unión en lados opuestos probablemente estarán sujetas a momentos flexionantes de signos opuestos. Los factores más importantes a considerar en el diseño de uniones viga-columna incluyen:

1. Cortante.
2. Anclaje del refuerzo.
3. Transmisión de carga axial.

4. TIPOS DE UNIONES EN MARCOS DE CONCRETO REFORZADO

4.1 Según su Configuración Geométrica

De acuerdo al tipo de anclaje de las varillas de las vigas, las uniones se pueden clasificar en interiores (las varillas pasan rectas a través de la unión, Fig. 3) y en exteriores (las barras se anclan mediante ganchos, Fig. 4). Atendiendo a la configuración de los elementos adyacentes, existen varios tipos de uniones exteriores. Por claridad en el dibujo no se muestran las losas de piso (monolíticas con las vigas).

4.2 Según el Intervalo de Comportamiento

Aunque es preferible diseñar las uniones para que permanezcan en el intervalo elástico, es muy posible que ocurran deformaciones inelásticas en ella si los elementos adyacentes, vigas o columnas, se deforman plásticamente. En este caso las deformaciones inelásticas a lo largo de las varillas penetrarán la junta; esta unión será del tipo inelástico. Por otro lado, es posible diseñar un marco de manera que se articulen las vigas lejos de la unión (Fig. 5), de manera que esta permanezca en el intervalo elástico. Para este tipo de marcos la unión será elástica.

5. MECANISMOS DE RESISTENCIA AL CORTE EN UNIONES INTERIORES

Un marco de concreto reforzado, diseñado según el Reglamento de Construcciones para el Distrito Federal (RDF-87) debe disipar energía ante cargas inducidas por sismos, mediante la formación de articulaciones plásticas en las vigas. Cuando éstas desarrollan sus resistencias máximas, las uniones estarán sujetas a fuerzas cortantes elevadas.

Bajos los efectos sísmicos, se generan momentos flexionantes y fuerzas cortantes en vigas y columnas que esfuerzan al núcleo de la unión como se ilustra en la Figs. 6a y 6b. En este dibujo las resultantes de los esfuerzos de tensión se denotan como T, y las resultantes de esfuerzos de compresión en el concreto y acero se identifican como C.

Por equilibrio de fuerzas horizontales tenemos que

$$V_{jh} = T_B + T'_B - V_c$$

donde V_c es el promedio de las fuerzas cortantes de las columnas superior e inferior. El cortante en columnas es el correspondiente al desarrollo de los momentos máximos en la vigas.

Se han identificado dos mecanismos de resistencia al corte en uniones interiores. El mecanismo del puntal diagonal de compresión (Fig. 6c) se forma a lo largo de la diagonal principal de la unión como resultante de los esfuerzos verticales y horizontales de compresión que actúan en las secciones críticas de vigas y columnas. Es importante notar que el puntal se desarrolla independientemente de las condiciones de adherencia de varillas dentro de la unión. En este mecanismo, el nudo fallará cuando el puntal lo haga por compresión-cortante. En la Fig. 7 se muestran los lazos histeréticos de una unión que falló por cortante.

En el segundo mecanismo, llamado de armadura, se forman pequeños puntales diagonales distribuidos en la unión (Fig. 6d). Estos puntales deben ser equilibrados por esfuerzos de tensión en el refuerzo vertical y horizontal, y por esfuerzos de adherencia a lo largo de las barras de vigas y las varillas externas de la columna. Este mecanismo es posible únicamente si se mantiene una buena adherencia a lo largo del refuerzo de vigas y columnas. Sin embargo, es difícil mantener una buena adherencia después de la fluencia del acero. Conforme la adherencia se deteriora (y, por tanto, las varillas se deslizan dentro de la unión) el mecanismo de la armadura se degrada, de manera que el puntal diagonal de compresión tiene que resistir la mayor parte de las fuerza cortantes en la unión.

Las expresiones de diseño de las Normas Técnicas Complementarias de Diseño y Construcción de Estructuras de Concreto (NTC-Concreto) se basan en el mecanismo del puntal diagonal a compresión.

Ensayes de laboratorio han indicado que la resistencia al corte de uniones aumenta con la resistencia del concreto.

Por otro lado, se ha mostrado que es necesario colocar una cantidad mínima de refuerzo transversal para mantener el concreto y la resistencia al corte. También se ha notado que si se incrementa la cantidad de acero lateral en la unión, no se obtienen mayores resistencias al corte.

Los resultados de ensayes indican que la presencia de vigas transversales, sean cargadas o no, mejoran el comportamiento de la unión porque contribuyen a preservar la integridad del concreto del núcleo. En forma similar, el ancho de las vigas también influye en el comportamiento.

Se han aplicado diferentes niveles de carga axial en la columna en ensayes de laboratorio. Los resultados indican que la carga axial en la columna no influye en la resistencia de la unión.

La losa de piso ha sido incluida en alguno especímenes. De acuerdo a lo observado, se ha destacado la participación de la losa en el confinamiento de la unión y en la capacidad a flexión de las vigas. Se ha encontrado que la contribución del refuerzo de la losa sumado a aquélla del acero de la viga, aumenta la resistencia a momento negativo (lecho superior a tensión).

Se ha encontrado que un deficiente comportamiento de la adherencia afecta severamente la rigidez y capacidad de disipación de energía de la unión. Aun más, el deterioro en la adherencia modifica el mecanismo de transmisión de fuerza cortante. En la Fig. 8 se presentan las respuestas histeréticas para dos modelos, llamados A y O. Para el espécimen A se aprecian lazos con baja disipación de energía (área interna reducida) y deterioro de la rigidez debidos a un anclaje inadecuado de las barras rectas, mientras que las curvas para el modelo O exhiben una respuesta estable con buena disipación de energía.

Los parámetros que influyen en la adherencia de las varillas a través de las uniones son:

1. Confinamiento, que afecta significativamente el comportamiento de la adherencia bajo condiciones sísmicas. La adherencia de la barras de vigas puede mejorarse si se aumenta el confinamiento, ya sea mediante una mayor carga axial o por medio del refuerzo longitudinal interior de la columna.
2. Diámetro de la varilla. Aunque no afecta significativamente la resistencia a la adherencia, sí limita la fuerza máxima que puede ser transferida por este mecanismo. Por tanto, la relación entre el diámetro de las varillas con respecto a las dimensiones de la unión debe mantenerse constante (límite superior). Mientras mayor es esta razón, aumenta la probabilidad de falla de la adherencia.
3. Resistencia a la compresión del concreto. No afecta de manera importante ya que la adherencia depende de la resistencia a la tensión del concreto.
4. Separación entre las varillas. Si la separación es menor de cuatro veces el diámetro de la varilla, la resistencia de adherencia disminuye en un 20%.
5. Tipo de corrugación. La reacción de la corrugación contra el concreto circundante es la fuente más importante de la adherencia. Debe considerarse la posición de las varillas durante el colado. En efecto, si se colocan 30 cm o más de concreto por debajo de la varilla, la resistencia a la adherencia disminuye.

6. UNIONES EXTERIORES VIGA-COLUMNA

Puesto que en una unión exterior sólo se conecta un viga a la columna, en la dirección de estudio (Fig. 9), la fuerza cortante en la unión será menor que la que se aplica en uniones interiores de dimensiones y refuerzo iguales. De las resultantes del dibujo, la fuerza cortante horizontal en la unión es igual a

$$V_{jh} = T - V_{col}$$

Análogamente a las uniones interiores, se distinguen dos mecanismos de resistencia al cortante: el del puntal diagonal de compresión y el de la armadura.

Con objeto de obtener un comportamiento adecuado de uniones exteriores ambos lechos de las varillas de las vigas deben doblarse hacia la unión; el gancho debe colocarse lo más cerca de la cara externa de la columna como sea posible, a menos que la columna sea muy profunda (ya que sería un muro esbelto).

De acuerdo a la gráfica, para resistir los momentos flexionantes y las fuerzas cortantes sísmicas, se formará un puntal diagonal (ver Fig. 9) entre el radio del doblez de la varilla superior y la esquina inferior derecha de la unión. Es importante destacar que para mantener este mecanismo de transferencia de carga es indispensable confinar la unión con refuerzo transversal.

En efecto, el acero lateral en uniones exteriores persigue dos objetivos. Primero, confinar el concreto a compresión para incrementar su capacidad de deformación y mantener su resistencia (quizá aumentarla). Segundo, confinar el tramo recto del gancho que tratará de salirse por la cara externa de la columna.

Los siguientes aspectos deben considerarse en el diseño de uniones exteriores:

1. Si se espera la formación de una articulación plástica en la cara de la columna, el anclaje de las varillas de la viga se debe suponer que inicia dentro de la columna. Las NTC-Concreto suponen que la sección crítica, a partir de la cual se mide la longitud de desarrollo, coincide con el paño externo del núcleo de la columna.
2. Para garantizar un anclaje adecuado de las varillas de la viga en columnas poco profundas se recomienda:
   a. Usar varillas de diámetro pequeño.
   b. Emplear placas de anclaje soldadas a las varillas.
   c. Colocar pequeñas varillas en el radio interior del doblez para retrasar el aplastamiento o desprendimiento del concreto en ese lugar.
   d. Colocar una cantidad suficiente de estribos horizontales para restringir el movimiento del gancho.
3. Las varillas de las vigas deben doblarse hacia dentro de la unión. El detalle de colocar el doblez hacia afuera de la unión, es decir, hacia la columna, no es adecuado en zonas sísmicas.
4. Colocar el doblez del gancho lo más cercano a la cara externa de la columna.
5. Cuando la arquitectura del edificio lo permita, o cuando vigas peraltadas lleguen a columnas esbeltas, se recomienda terminar las varillas de las vigas en pequeñas extensiones en la fachada (Fig. 10). Este detalle mejora notablemente las condiciones de anclaje de las varillas, lo que se traduce en un comportamiento superior de la unión.
6. Para reducir los esfuerzos de adherencia, siempre es preferible el empleo de varillas con el menor diámetro como sea práctico. En uniones exteriores, no es aplicable el requerimiento del diámetro de la varilla en función de las dimensiones de la columna. En general, es más fácil cumplir con los requisitos de anclaje en las uniones exteriores que en las interiores.

7. DISEÑO DE UNIONES VIGA-COLUMNA SEGUN NTC-CONCRETO

Como se mencionó anteriormente, el diseñador debe prestar atención a dos aspectos para lograr que la unión viga-columna se comporte adecuadamente: la resistencia de la unión al corte y el anclaje de las varillas de la viga. Ambos estados límite son cubiertos por los requisitos para el diseño de uniones viga-columna de NTC-Concreto incluidos en el Art. 5.4. Para revisar la resistencia del nudo a fuerza cortante en cada dirección principal, en forma independiente, se considerará un plano horizontal a media altura del nudo. Para esta revisión, las NTC-Concreto distinguen dos casos:

1. Nudo confinado, que es aquél al cual llegan cuatro trabes y en donde el ancho de cada una es al menos 0.75 veces el ancho respectivo de la columna.
2. Otros nudos, que son los que no satisfacen lo anterior.

Para nudos confinados el esfuerzo máximo nominal es igual a $5.5\sqrt{f^*_c}$ y para otro tipo de uniones es igual a $4.5\sqrt{f^*_c}$. Se supone que un nudo confinado resiste cargas superiores a otros nudos. El esfuerzo nominal obrará sobre un área definida por la profundidad de la columna y un ancho efectivo, que es, por lo general, igual al promedio de los anchos de la o las vigas y de la columna en la dirección de análisis. Los esfuerzos máximos señalados se refieren a la resistencia a compresión-cortante del concreto en el mecanismo del puntal diagonal de compresión.

Para confinar el concreto del nudo, así como los ganchos de las varillas en uniones exteriores, se debe colocar refuerzo transversal mínimo como en columnas. Al igual que en columnas, el acero lateral estará formado por estribos cerrados de una pieza, sencillos o sobrepuestos. Los estribos deben rematarse con dobleces de 135 grados, seguidos de tramos rectos de no menos de 10 diámetros de largo. La separación no debe exceder de la cuarta parte de la menor dimensión transversal del elemento, ni de 10 cm. Los requisitos anteriores buscan preservar la integridad del concreto de manera que se mantenga el mecanismo de transferencia del puntal diagonal de compresión. Es importante cumplir estrictamente con los requisitos anteriores para evitar daños severos, difíciles de reparar, en la unión. Si el nudo está confinado como se indicó arriba, las NTC-Concreto permiten usar la mitad del refuerzo transversal mínimo. Esto reconoce que las trabes que llegan a la unión la confinan por lo que no se requiere la misma cantidad de acero lateral que para nudos no confinados.

El segundo aspecto importante en el diseño es el anclaje de las varillas. Para evitar fallas de adherencia que, como ya se dijo, afectan la rigidez y capacidad de disipación de energía de la unión, las NTC-Concreto señalan que el diámetro de las barras de vigas y columnas que pasen rectas a través de la junta deben seleccionarse de manera que la dimensión del elemento en la dirección de la varilla sea cuando menos 20 veces el diámetro de ella. Así, por ejemplo, si la columna tiene una profundidad de 50 cm, el diámetro máximo de varilla que puede usarse es del No. 25 (la designación esta en milímetros y corresponde a una varilla de 1 pulg de diámetro). Se permite que el peralte de la viga sea 15 veces el diámetro cuando más del 50% de la carga lateral es resistida por muros estructurales o cuando la carga axial de la columna superior al nudo es alta.

Para juntas exteriores, como se explicó, las varillas deben terminar en ganchos a 90 grados. La longitud de desarrollo se mide desde el plano externo del núcleo de la columna hasta el paño externo de la barra en el doblez. La ecuación para calcular la longitud de desarrollo en kg y cm^2 es

$$L_h = 0.076\ d_b \frac{f_y}{\sqrt{f_c'}}$$

donde d_b es el diámetro de la varilla. El tramo recto después del doblez no será menor de $12d_b$.

Un último aspecto relevante con uniones viga-columna es que las varillas longitudinales de las vigas deben pasar por el núcleo de la columna. Esto se debe a que no existe suficiente información experimental que permita extrapolar los requisitos de diseño vigentes a casos en los que el refuerzo pase o se ancle afuera de la columna. El refuerzo de los modelos ensayados que sirvieron de base para la formulación de los requerimientos de las NTC-Concreto se anclaba o pasaba a través del núcleo.

8. UNIONES DE ESQUINA

Las uniones de esquina, o de rodilla, empleadas en marcos planos o en otras estructuras, presentan un problema especial de detallado. La esquina puede estar sujeta a fuerzas que la traten de abrir o de cerrar (Fig. 11). Una situación similar ocurre cuando dos muros se intersecan. Desafortunadamente no existe suficiente información sobre el comportamiento de este tipo de uniones y del las del siguiente tipo antes cargas cíclicas. La discusión que sigue es válida para cargas monótonas.

Ensayes de laboratorio han indicado que la falla de este tipo de uniones es el resultado del agrietamiento por tensión diagonal, por falla de anclaje (sobre el refuerzo), por fluencia del acero, por daño del anclaje o por aplastamiento del concreto. Aunque la tensión diagonal es a menudo ignorada, esta puede ser la causa de la falla en esquinas que se abren. El anclaje es el problema más común en esquinas que tratan de cerrarse. El agrietamiento diagonal en una unión que se abre se presenta en la Fig. 12. El refuerzo debe colocarse como se ilustra. El refuerzo se debe anclar con ganchos. Para confinar el concreto que resiste compresión se deben colocar estribos cerrados.

9. UNIONES EN FORMA DE "T"

Este tipo de uniones se presentan cuando una losa se conecta con un muro, cuando los muros se conectan con zapatas, en vigas que llegan a columnas, o en columnas que llegan a vigas de techo. Se han conducido ensayes de laboratorio (Fig. 13) en los cuales se observó un incremento en la resistencia conforme se usan los detalles de las gráficas de la derecha. En efecto, solamente con cambiar la dirección del gancho se incrementó la resistencia en 40%. Esto se debe a que el puntal de compresión que se desarrolla a través de la junta produce el desconchamiento del concreto abajo del gancho cuando éste se dobla hacia la derecha. Cuando el gancho se coloca hacia la izquierda, el puntal de compresión reacciona contra el radio del doblez, como en el caso de uniones exteriores viga-columna.

10. LAS UNIONES EN LOS PLANOS DE CONSTRUCCION

Para evitar errores o malas interpretaciones durante la construcción de uniones, es necesario que el diseñador muestre los detalles de las conexiones en los planos estructurales. Al momento de incluir estos detalles, el diseñador es forzado a verificar que dicho detalle se pueda construir. Esto se relaciona con la colocación del refuerzo, y con la colocación y compactación del concreto. Por ejemplo, una viga del mismo ancho que el de la columna causará problemas al obrero de la construcción si durante el diseño no se consideró que un recubrimiento igual sobre el acero transversal de la viga y la columna, provocará que los refuerzos longitudinales de la columna y la viga coincidan. Si en este caso la sección transversal de la columna se agrandara no habría problema. Las NTC-Concreto requieren que se incluyan dibujos acotados y a escala del refuerzo en uniones viga-columna (Art. 5.4); sin embargo, lo anterior debe ser aplicado en otro tipo de uniones.

11. BIBLIOGRAFIA

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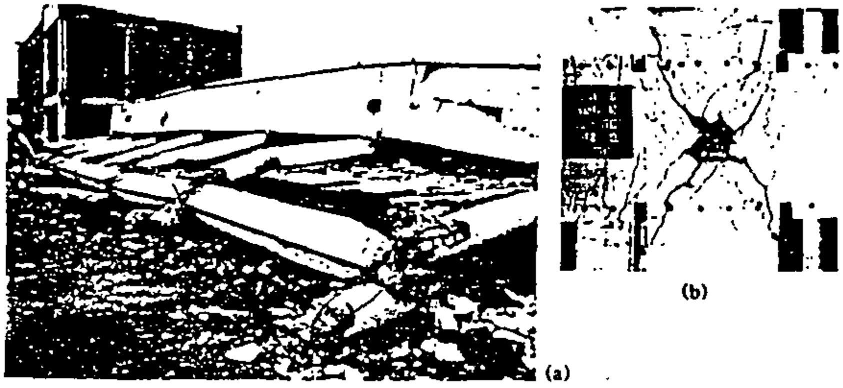


Fig. 1 Falla de uniones viga-columna en (a) el sismo de El Asnam y (b) en un espécimen de laboratorio

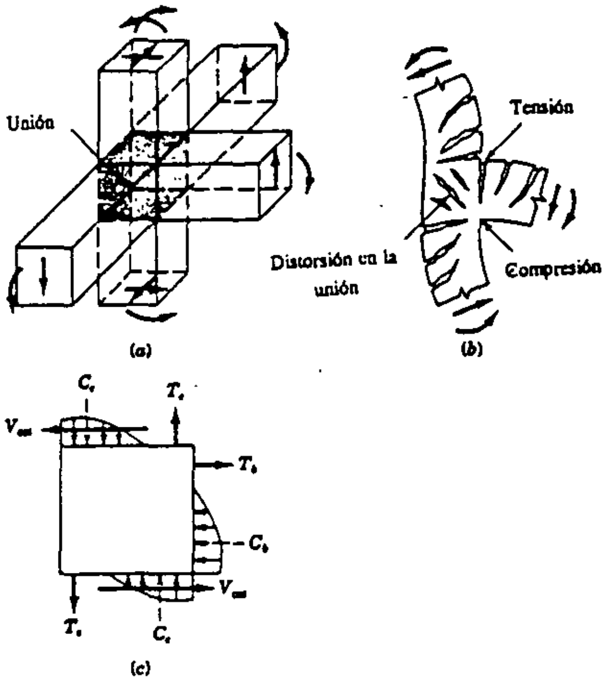


Fig. 2 Fuerzas y distorsión de una unión viga-columna

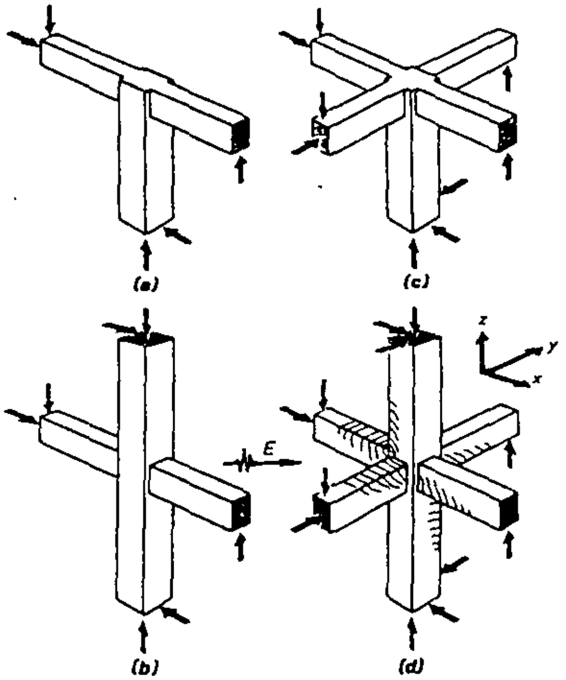


Fig. 3 Uniones interiores viga-columna

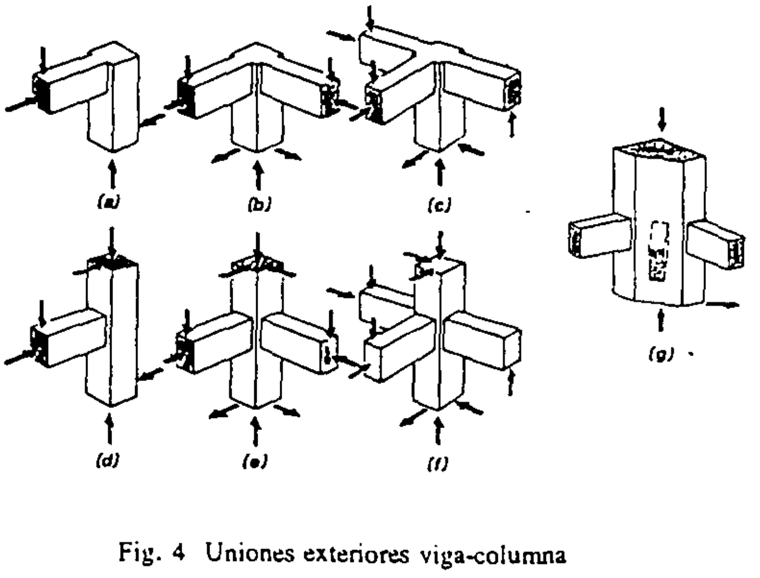


Fig. 4 Uniones exteriores viga-columna

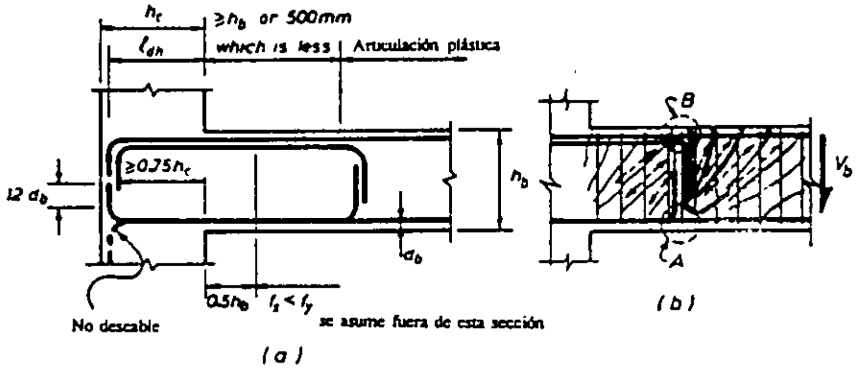
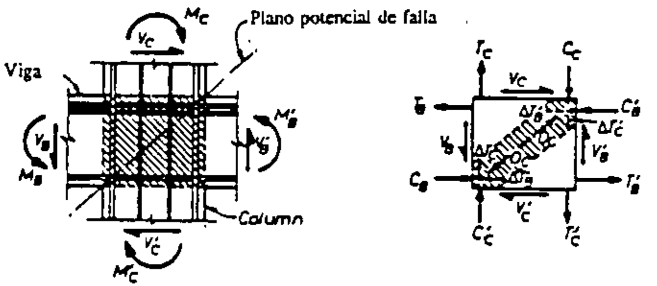
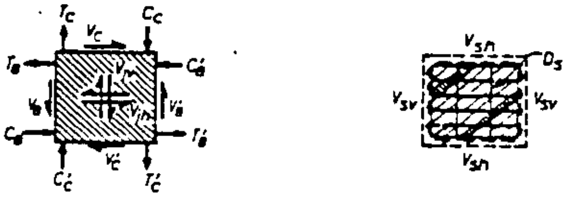


Fig. 5 Relocalización de la región de articulación plástica lejos de la cara de la columna



a) Acciones externas en un plano de unión de marco

c)Transferencia de cortante por un mecanismo de puntal en compresión diagonal



b)Resultantes de esfuerzos internos en el nucleo de la unión

d)Transferencia de cortante por un mecanismo de armadura de las fuerzas de adherencia del refuerzo a través de la unión

Fig. 6 Mecanismos de resistencia al corte en uniones interiores viga-columna

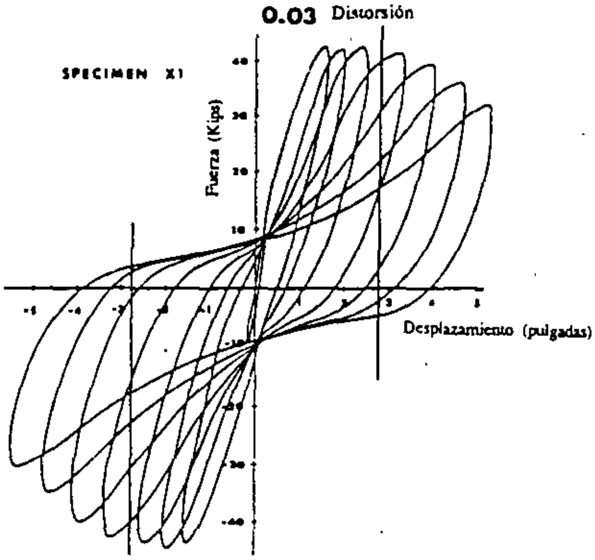


Fig. 7 Falla por cortante en una unión interior viga-columna

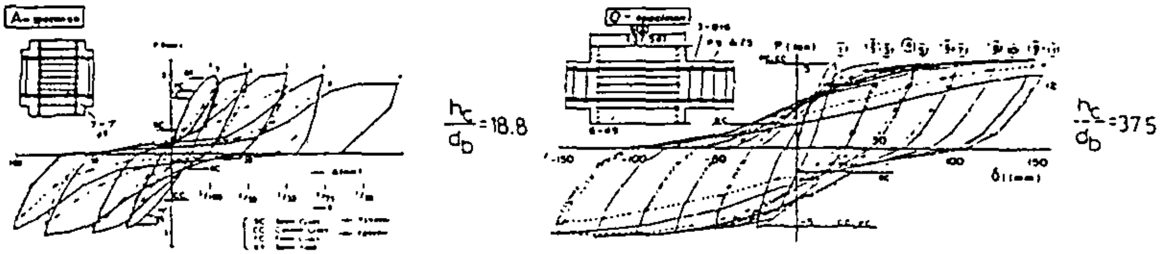


Fig. 8 Influencia del anclaje en la respuesta histerética de una unión interior viga-columna

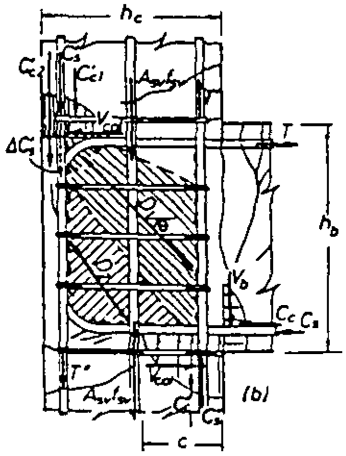


Fig. 9 Mecanismos de resistencia al corte en uniones exteriores viga-columna

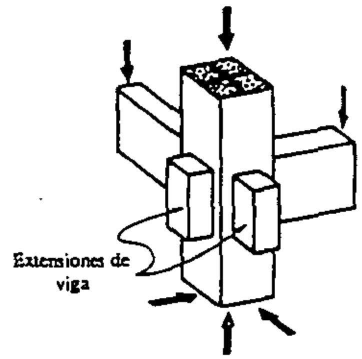


Fig. 10 Extensiones de viga para colocar las varillas

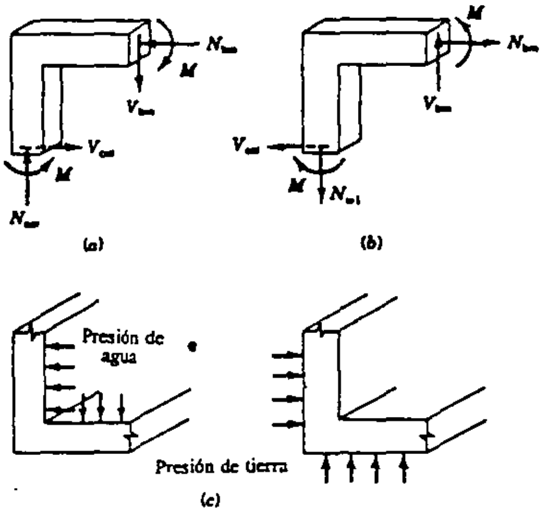


Fig. 11 Uniones de esquina

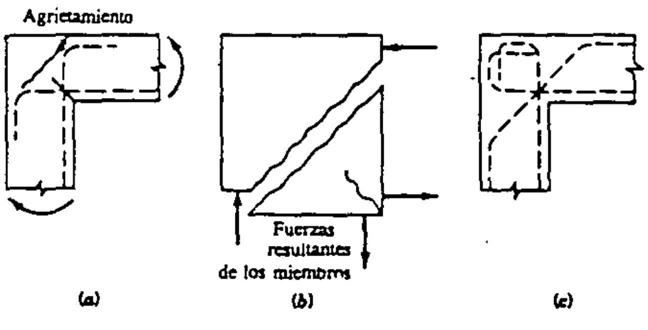


Fig. 12 Unión de esquina que se trata de abrir

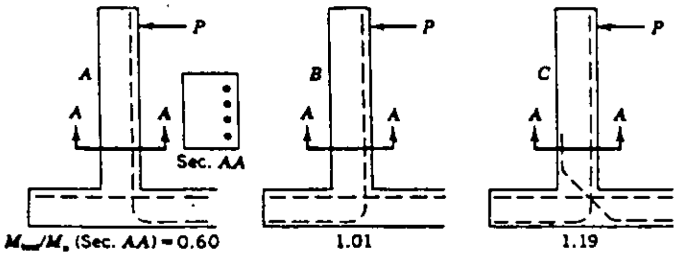


Fig. 13 Uniones en "T"

L_c^R = Actual distance between critical regions where M_{max}^R developed

L_c^C = Nominal clear height of the columns assumed in the analysis and design of column

Φ_V^c = Code strength reduction factor for shear

$\Phi_{(M,P)}^C$ = Code strength reduction factor for columns

$(b_w d)^R$ = Actual size of columns

$(b_w d)^C$ = Assumed size of columns used in code equations for estimating v_c^C

In eq. 1, the M_{max}^R / M_u^C can be expressed as:

$$\frac{M_{max}^R}{M_u^C} = \frac{f_{s\ max}^R}{f_y^C} \; F\left[\frac{P^R \ \& \ M_{max}^R}{P^C \ \& \ M_u^C}\right] \tag{2}$$

where:

$f_{s\ max}^R$ = Maximum steel stress that can be generated

f_y^C = Code specified yield strength

P^R = Actual axial load action on section where M_{max}^R is generated

P^C = Axial load estimated based on code force acting on section where M_u^C is computed

Considering that $f_{s\ max}^R$ can be up to twice f_y^c and $F\left[\frac{P^R \ \& \ M_{max}^R}{P^C \ \& \ M_u{}^C}\right]$ can also be considerably larger than one, either because P^R is a compression force smaller than the P corresponding to the balanced point but larger than the assumed or estimated P^C or because P^R can be a tension force decreasing the actual shear resistance, the value of the M_{max}^R /M_u^C can be larger than two. Additionally, because of the actual effects of lap splicing and/or nonstructural elements, the value of L_c^R / L_c^C could be larger than one, the Φ_V^c / $\Phi_{(M,P)}^c$ is 0.85/0.70 (ACI 318-71), and $(b_w d)^R$ / $(b_w d)^C$ can be larger than one.

is equivalent to only about to 50 psi (0.35 MPa), thus sliding shear distress is possible in columns subjected to tension. For a given interstory relative displacement of, say 0.5 in. (13 mm), the compression columns would also have to distort the same amount and could undergo damaging sliding shear displacements. A slip of less than about 0.05 in. (13 mm) is already harmful; such a slip is quite conceivable considering the relative magnitudes of approximate sliding shear and column lateral stiffnesses [72].

All the available data have been obtained under excitations which produce compressive axial forces, shear, and bending in only one main plane of the element. Since columns are usually subjected to biaxial shear and bending, there is an urgent need for experimental and analytical studies of the inelastic behavior of columns under the combined effects of axial force and biaxial shear and bending. Also, during extreme earthquakes tension forces can be developed due to overturning moments and the vertical component of acceleration. Therefore it is of paramount importance to carry out studies on columns in which the axial force is varied from compression to tension. To the best of the author's knowledge, the only experimental work on the effect of varying axial force in columns has been carried out in Japan [142] and more recently at the University of Texas. These studies will be discussed later.

The columns are still the elements most susceptible to failure in destructive seismic ground motions. This has been demonstrated by inspection of damage in many recent earthquakes. In Ref. 93 Aoyama discusses the causes of shear failure of columns and countermeasures taken in Japan. The causes of such failure can be found, the author believes by: (1) Analyzing present methods of evaluating column action during severe earthquakes, and estimating the range of demands placed on columns; (2) Studying the sensitivity of the nominal unit shear stress to the main factors involved in its computation; (3) Comparing this sensitivity with the values specified by present seismic codes.

Park [25] and Paulay [47] discuss the problems encountered in estimating the column actions and therefore in formulating a design procedure that will give an acceptable degree of protection against undesirable behavior.

To illustrate the sensitivity of the nominal unit shear stress to the main factors involved in its computation the following equation compares the probable realistic value of the maximum nominal unit shear shear stress, v^R_{max}, with the value given by ACI or UBC codes v^C_u [35, 36].

$$\frac{v^R_{max}}{v^C_u} = \frac{M^R_{max}}{M^C_u} \; \frac{L^R_c}{L^C_c} \; \frac{\Phi^C_V}{\Phi^C_{(M,P)}} \; \frac{(b_w d)^R}{(b_w d)^C} \qquad (1)$$

where:

M^R_{max} = Maximum moment that can be developed in the real column

$M_u{}^C$ = Ultimate moment capacity computed according to code provisions column philosophy. However, these deformation capacities may be insufficient when compared to the magnitude and nature of deformation demands that may be expected in frames designed with soft stories. Furthermore, the above observations are valid for cases where there is essentially no fluctuation in axial force. The change from a ductile shear-compression failure mode in columns with certain axial compressive force to a brittle diagonal tension mode in similar columns in which the axial load decreased, suggests the need to investigate the inelastic behavior of short columns in which the axial force varies. The axial force should be varied with shear reversal from a maximum compression to either a tension value or a smaller compression.

(2) A comparison of analytical and experimental shear strengths indicates that code shear capacities are adequate if actual mechanical characteristics are used. However, if the expected inelastic deformations are higher than those used in the tests, these code provisions may be inadequate. The concrete degradation associated with large inelastic cyclic deformations will result in an entirely different state than that on which the code recommendations are based. To develop such large deformation and still maintain shear strength, the contribution of concrete should be ignored unless the core concrete can be kept effectively confined, even under the largest deformation.

(3) Comparison of the behavior of columns subjected to different deformation histories demonstrates that cyclic deformation reduces the maximum inelastic deformation a member can experience in a given direction. This fact should be kept in mind when design is controlled by inelastic deformation demands. It will be necessary to specify not only the deformation level that is expected, but also the number and type of reversals (partial, full) expected. The magnitude of the nominal shear stresses developed in some of the columns tested show that moderate ductile behavior and high shear stresses are compatible. However, it is necessary to provide sufficient and properly detailed transverse reinforcement.

(4) A comparison of the behavior of columns with different types of transverse reinforcement indicates that the circular spiral is more effective in maintaining a member's shear strength. Its continuity and relatively close spacing provide excellent confinement for the core concrete and restrain the width of inclined shear cracks. However, the close spacing of the spiral, and the fact that it is responsible for significant spalling through the height o of the column, reduces the area of concrete in concrete with the longitudinal reinforcement and thus contributes to bond deterioration along this reinforcement.

4.2.4 Analytical Prediction of the Hysteretic Behavior of Columns. Many attempts have been made to predict the hysteretic behavior of columns, starting from the mechanical characteristics of the materials used and following the classical approach of continuous mechanics. In Ref. 88 Zagajeski and Bertero discuss different methods and models that have been used, and the difficulties encountered in predicting the hysteretic behavior. Perhaps an easier approach is to directly model the load-deformation relationship as was done by Atalay and Penzien for flexural members subjected to high shear [95, 96]. This was done also by others [98] using a degrading trilinear model for the restoring force-deformation characteristics of reinforced concrete structures failing primarily in flexure. Jirsa in Refs. 28 and 97 reviewed the analytical work done in modeling behavior of columns and classified these different models in two categories: "conceptual model" and "element or filament model". An example of good agreement obtained by using conceptual models is shown in Fig. 17(a).

Therefore the real nominal unit shear stress, v_{max}, can be four or more times than the value obtained using code procedure v_u^C ; the value for which the shear reinforcement was designed. Thus it is not surprising that many of the frame failures observed have been due to shear failure of the columns. Therefore ther is a need to conduct statistical studies of the value of v_{max}^R / v_u^C in existing buildings.

4.2.2 Experimental Research and Development in Japan. Ohmori in Ref. 84 points out studies in Japan where the behavior of columns was investigated in order to improve the design and construction of real R/C structures. Among the important experimental findings of the research, the following deserve special mention.

Newly Developed Transverse Reinforcements--The three types of lateral reinforcement shown in Fig. 16 (a), typical of present construction, were tested, considering three levels of reinforcement ratio for each type. The results reviewed in Ref. 16 and summarized in Fig. 16(b) show the advantages of tied (type A) and spiral (type S) columns when compared with hooped columns (type J). However construction of tied columns presents some difficulties. Therefore, new types of transverse reinforcements that could offer good confinement and could be easily fabricated were sought [142]. A combination of spiral and hoop reinforcements was developed which was named the KS type. It showed ductile and stable hysteretic behavior similar to that observed for tied columns (Fig. 16[b]). Figure 16(c) shows the different types studied and Fig. 16(d) shows the results obtained. It is clear that the KS type columns showed the most stable and ductile behavior. The other two types which showed good behavior for the same reinforcement ratio, were the spirals (S) and the tied (T) arrangements.

Splices of Large Size Re-Bars--In the lower story columns of tall buildings it becomes necessary to use large size rebars. In this case the use of lap splices offers serious difficulties. Various types of welded and sleeve joints were tested. Among the most effective in the sleeve category were the squeezed joint, and the Caldwell joint [94].

Very little information is available on the behavior of lapped splices. As pointed out by Gergely [72] impact type tests of splices showed an increase in splice capacity and stirrups enhance the toughness and ductility of splices. Research is needed on the behavior of lapped splices and mechanical splices at high-level load reversals.

4.2.3 Concluding Remarks Regarding Experimental Studies. From analysis of results obtained in different investigations up to date it can be concluded that:

(1) Short R/C columns, if designed and detailed to satisfy code recommendations for ductile moment-resisting frames [36] can develop moderate inelastic deformation prior to either a brittle shear failure or significant shear degradation, when subjected to high constant axial loads and to cyclic shear reversals. The word moderate should be emphasized. It is felt that the inelastic deformation capacities found in the investigations (particularly in Ref. 88) would prove adequate when compared to the magnitude and nature of inelastic deformation demands that may be expected for columns that are components of frame systems designed on the basis of a weak girder-strong

"2D excitation of single mass systems produces a greater period shift, which in turn can lead to larger displacement response, depending to some extent on the initial system period. Gravity loads acting through the increased lateral displacements may cause collapse. Although details of input motion and shape of hysteresis curve play a role, they do not appear to decisively influence the general trends. The combined effect of correlation of the orthogonal components of response and of inelastic interaction generally appears to increase with relative strength of the excitation. 2D ductilities about twice as large as 1D ductilities are typical at 1D ductilities of about 5 or more.

Since the effect of gravity load is consistent, an examination of responses without the P-δ effect is sufficient to indicate possible problems. Two criteria are useful for this prupose: 1D ductility demand and system period. The most important indicator is the 1D ductility demand calculated from a one-dimensional inelastic response analysis. If the system strength is sufficient to restrict the 1D ductility demand to about two, no difficulties should occur. In conjunction with this, however, the system period should be taken into account, since the consequences of a slight underdesign are more serious for short period (stiff) systems than for long period (soft) systems.

Frames resisting seismic loads in both horizontal directions should be designed so that column deformations do not substantially exceed "yield". An important factor not accounted for by response studies of single mass systems is the distribution of inelastic deformation between girders and columns in space frames, This distribution is different for 2D motion than for 1D motion, since columns may yield sooner in 2D motion. The few results available for multi-story structures indicate that 2D motion increases column response ductility demand and decreases girder response ductility demand. While a varying axial load does produce large changes in the lateral restoring force-deformation characteristics of a single column, when these characteristics are averaged over several columns in a story, the effect on the total lateral force-deformation resistance curve for the story appears to be slight. The influence of ground motion characteristics should be more thoroughly explored. Besides duration and general intensity level of the excitation, the relative strength of all components is important. Extensive work remains to be done along these lines."

Jirsa, et al. presents a thorough review not only of analytical work in this field but also of the experimental studies that have been conducted until 1978 [97]. In reviewing the analytical work, Jirsa, et al., classified the proposed models in two categories: conceptual; and element or filament models. They then summarized the models, applications, advantages, and disadvantates. One of the main problems in conducting experimental studies is the selection of realistic loading histories. The problems discussed for 1D models are increased considerably because of the many possible combinations of the path of the two components.

4.3.2 Experimental Studies. These studies can be classified as studies of flexural behavior and shear behavior [97].

4.3.2.1 Flexural Behavior. Takizawa and Aoyama [98] conducted some experiments and compared their test results with analytically predicted values on a conceptual model. Measured and analytically predicted response for unidirectional and 2D loading histories are shown in Fig. 17: The measured and analytical responses for the square (or diamond) loading history (Fig. 17[b])

In 1977, after reviewing analytical studies, Gergely [72] concluded:

(1) Many researchers have used various types of idealizations and hysteresis rules in nonlinear analyses and have shown that good results can be obtained when the idealizations directly correspond to the system being modeled. In most cases, however, not all modes of stiffness deterioration were included in the analysis and in the corresponding tests. Significant advances have been made in system identification techniques that allow the determination of stiffness properties from test results, or enable linearization of non-linear systems. Most nonlinear analyses are too complex for design use but they are helpful in identifying the effects of various factors as well as in aiding in the planning of test programs.

(2) Many factors affecting nonlinear response have not yet been isolated or studied sufficiently. Therefore, most analyses are reasonably accurate only for the test program for which they were derived. If other factors modify the behavior or if a different type of loading is applied, the agreement between analysis and test is generally poor, especially after two or more load cycles.

4.3 Hysteretic Behavior of Columns under Three Dimensional Loading

An earthquake ground motion at the foundation of a structure has six simultaneously acting components: three translational and three rotational. Thus, the columns in a space frame are subjected to three dimensional (3D) loading components, which will vary with time during dynamic response to the ground motion. This is particularly true in the case of exterior columns in a space frame. In the case of interior columns the variation of axial force during the earthquake might not be important and therefore, although there is a state of 3D-loading, only the biaxial bending and shear will vary with time. It is common to refer to this as a case of biaxial bending or two dimensional (2D) behavior, although strictly speaking a 3D-state of loading exists even for a 2D ground motion.

In 1972 the author pointed out the lack of data regarding the 3D behavior of columns [16]. As a consequence of damage from the 1968 Tokachioki and 1971 San Fernando Earthquakes, several studies were conducted to see if it was possible to analytically predict such damage. Most of these analytical studies were based on the conventional planar behavior of the structural elements. Since these studies did not sucessfully justify the observed damage, and since there was evidence of biaxial bendings in certain columns, particularly in the case of the main buildings of the Olive View Hospital, analytical studies of the effects of 2D ground motions were started.

4.3.1 Analytical Studies on the Effects of 2D Ground Motions and Biaxial Loading on Columns. Japanese and American researchers have conducted a series of analytical studies on the effects of 2D ground motions on columns. The analytical work of Takizawa and his associates in Japan [99] and Pecknold and his associates in the U.S. [100] deserves special mention.

In Ref. 99 Takizawa concludes that "The margin of safety against collapse of R/C structures is very small when the effects of biaxial response, deteriorating ductility, and gravity are all combined. In Ref. 100 Pecknold and Suharwardy review the analytical work conducted until 1977 and summarize the findings as follows:

on the effect of high shear on columns [97, 104]. The primary variable is the loading history. The geometry of the specimens is the same for all tests. The column is a stiff element (12 in. square, 36 in. long) framing into fixed ends representing a stiff floor system. Two series of tests, (one with no axial force and the other with varying axial load), have already been carried out and reported by Jirsa and his associates [97, 104].

2D Behavior - No Axial Load Figures 18(a) and 18(b) compare the lateral force-deformation curves for two tests. In one test, the load history was applied in 1D, in the other it was applied in 2D following a square deflection path. The force-deformation relationships are shown for a principal axis of the column. Such a comparison indicates a severe reduction of capacity due to prior or simultaneous loading in the orthogonal direction. This is shown by the force orbit in Fig. 18(c), for the specimen subjected to a square load path. If, under 2D loading, the resultant force ($V_R = V_N^2 + V_{EW}^2$) is plotted against the resultant deformation or the radial deformation from the original position ($\Delta_R = \Delta_{NS}^2 + \Delta_{EW}^2$), differences between 1D and 2D response are not as large (Fig. 18[d]). Jirsa and his associates [97] pointed out that, while a great deal of additional testing will be needed to qualify the response, results to date indicate that 2D response "may be well correlated to resultant force-resultant deformation behavior regardless of the deformation path".

Otani, of the University of Toronto, Canada has recently started an experimental program to investigate the effects of 2D deformation on columns. He has reported the results from tests of two relatively slender columns (12 x 12 x 60 in.) [105]. Because of early fracture of the longitudinal reinforcement at the welding in a critical region, no data has been obtained under cyclic loading requiring large inelastic deformations. From the results obtained, Otani concluded that:

(a) An effect of biaxial lateral load reversals on the behavior of reinforced concrete columns was evident prior to the tensile yielding of longitudinal reinforcement;

(b) The effect of biaxial lateral load reversals was relatively small, in the specimens tested, after the tensile yielding of longitudinal reinforcement;

3D Behavior with Varying Axial Load As "mentioned above, more research on the effects of varying axial load on column behavior is needed." (research to date has been reported by Ohmori [84], Kokusho, et al. [106, 107], and Jirsa and associates [97]. The experiments in Japan were conducted under uniaxial bending; the work done by Jirsa was under biaxial bending. Jirsa concluded that while constant compressive loads had a slight influence, constant tensile loads had a greater influence on columns subjected to biaxial bending in comparison to an axially unloaded column subjected to biaxial bending. In particular, under cyclic biaxial bending, compressive loads increased the shear capacity slightly and tensile loads substantially reduced the stiffness of the column and the shear capacity at low load. However this reduced shear capacity did not deteriorate, even under large lateral deformation. Additional tests were conducted with 2D lateral loadings and axial load variation; however, the trends are not significantly different from those under constant tension or compression. Axial loads appear to have an influence on response only while the load is on the structure and do not influence subsequent response. This is quite different from lateral loading where loads in one direction influence subsequent response in the orthogonal are shown in Fig. 17(c). Note that the general shape of the measured curves is predicted by the analytical procedure. However, the magnitude of forces tends to differ, particularly at the largest deformation level, where the measured forces were considerably less than the predicted ones. This is apparent in the plot of the experimental and analytical force orbits shown in Fig. 17(d). The force orbit represents the locus of forces in the principal directions produced by the deflection orbit shown in Fig. 17(b).

Takiguchi and Kokusho [101], presented a summary of results from 26 specimens subjected to biaxial bending moments. The specimens were small, 10 cm. and 15 cm, square cross sections. The experimental results were compared with analytically predicted values using a finite filament model, and good agreement was found. Takiguchi and Kokusho concluded that "The influence of bending moment about one axis due to dead load on hysteretic characteristics about the other axis should be taken into consideration when conventional seismic resistant design methods (i.e. methods in which lateral forces are applied independently in two directional orthogonal to each other) are used for reinforced concrete columns."

Okada, Seki, and Asai [102] compared experimental results with the analytically predicted ones using a finite element model, and concluded that their analytical model simulated behavior reasonably well. As the severity of the 2D loading increased, the measured response clearly indicated the deterioration of strength and deformability of the columns.

Effect of Axial Load on Flexural Behavior As pointed out by Jirsa [97], the effect of axial load in the above studies was not significant. However, in the specimens tested the axial load was small or zero and remained constant throughout the 2D moment or lateral loading history.

4.3.2.2 Shear Behavior under 2D Loading. From the point of view of seismic resistant design, the ideal frame system would be one of which column hinging is prevented. This is not usually economically feasible. However, an acceptable degree of protection against premature yielding and excessive hinging should be attempted [20, 25, 47]. This design philosophy implicitly requires that shear failure be prevented or delayed so that the column may dissipate, by flexure yielding, an energy larger than that demanded by the most severe earthquake. This degree of protection against shear is not always easily achieved in practice, when columns are loaded in 2D.

As pointed out by Park [25], "The diagonal shear force resulting from biaxial bending in two-way frames due to concurrent seismic loading should be considered in design". The shear strength of rectangular column sections loaded along a diagonal has received little attention in the past. Tests have been conducted recently in New Zealand [103] on four reinforced concrete members with a 16 in. (406 mm) square section subjected to uniaxial or diagonal shear force and flexure with no axial load applied. Two arrangements of overlapping hoops were used. The difference between the diagonal shear strength and the uniaxial shear strength of identical specimens was zero for one pair and 3% for the other pair. The result is not surprising, since, although for diagonal shear the component of transverse bar forces in the direction of the shear force is smaller, the diagonal tension crack has a greater projected length and therefore intercepts more transverse bars: these effects compensate each other [25].

Jirsa and his associates have started an extensive experimental program

5. SEISMIC BEHAVIOR OF BEAM-COLUMN JOINTS

5.1 General

Efficient seismic resistant design may be achieved through predictions, or at least visualization of the structure's mechanical behavior under the excitations which it may be subjected to during its service life. To facilitate this prediction, the ideal would be to test real structures under such excitations. Since such tests are not economically feasible, basic structural components have been investigated separately. In the case of moment-resisting frames, the beams and columns have been investigated. Significant data on behavior have been obtained, and analytical methods of prediction have been formulated and used. Therefore the questions is: Can the response of the whole structure be predicted from the independent behavior of its components? Because of the interactions between these members, it is necessary to have information regarding the behavior of certain structural subassemblages. The author has discussed this problem in detail in Refs. 16 and 26.

Figure 19 illustrates the basic subassemblages of a moment-resisting frame whose behavior is essentially planar. Note that the beam-column joints are included and that there is distinction between the exterior and the interior beam-column joints. As will be discussed later, the actual subassemblages should be 3D and should consist of at least: a column; beams framing into the columns in two orthogonal directions; the joint between these two elements; and the floor slab they support. The behavior of these subassemblages should be studied under 3D loading conditions.

Because a failure of the joint means a failure of the column, ideally the joint should be the strongest and the stiffest element of the basic subassemblage. In the past this usually has been so. Surveys of earthquake damage usually show no evidence of joint failure, except in cases of very poor detailing and construction. However, because of numerous failures in beams, and particularly in columns, recent seismic codes have much more stringent requirements regarding design and detailing of these two elements. Therefore the author believes that the joint may become the weakest link in the subassemblage. This belief has been corroborated by recent experimental results in laboratories and in the field. In many cases, although there is no visible sign of distress in the joint, it has failed internally with a loss of the required anchorage to the main reinforcing bars of the beams and/or columns.

Current knowledge of the behavior of joints subjected to forces in one principal plane of a space frame is reviewed below. Following this is a more general discussion of the problem of joints in a space frame loaded in three directions.

5.2 Beam-Column Joints in Planar Frame System

In Ref. 16, the author made the following observations:

(1) Types of specimen: Subassemblages like those indicated in Fig. 19(a), where part of the floor slab is reproduced and gravity forces are applied through this slab, should be tested.

(2) Method of Testing: All tests must have a standard loading arrangement and sequence. The proper loading sequence can only be obtained by integrating analytical and experimental studies. The usual sequence of

direction. It should be noted that since columns were short, the P-δ effect was negligible.

4.4 Concluding Remarks

Although there have been many advances in understanding column seismic behavior most of these have been for columns under uniaxial bending and shear. Several analytical methods and models have been suggested for the prediction of real behavior of columns. However, most of these models are reasonably accurate only for the test program for which they were derived. If other factors modify the behavior or if a different type of loading is applied, the agreement between analysis and test is often poor, especially after two or more load cycles beyond yielding of reinforcement. Furthermore, most of the models are too complex for use in analysis or design practice. However, they are needed to do parametric and sensitivity studies, thus helping to: identify the importance of various factors; and aid in the planning of comprehensive experimental programs.

The Building Research Institute in Japan recently reported the result of 140 tests carried out during 1973-1976 [108]. "This report presents some of the most comprehensive information available on the behavior of R/C elements". Thorough analyses of this and other data will permit the improvement of present seismic design of columns.

Present seismic code provisions regarding detailing of columns appears to guarantee sufficient ductility to resist moderate demand of inelastic deformations if these take place in just one of the principal planes. However, during an earthquake a column can be subjected not only to biaxial bending but also to varying axial force. Although there have been some studies of these problems, there are many more factors influencing behavior for 3D than for 1D response. Therefore, it is not surprising that few advances have been made and that some of the results obtained do not, apparently, agree. It appears that bending and shear reversals in the two lateral directions increase the degree of stiffness deterioration under uniaxial bending. There can also be a significant decrease in strength and energy dissipation if the axial force can be a tensile force when large bending and shear exists. A practical solution to minimize the problems that tensile forces can create in columns has been developed by the Kajima Corporation [84, 94]. The outer columns of the first 5 stories of a modern 18 story building were post-tensioned.

No analytical model has been developed for predicting the behavior of columns under cyclic 3D loading inducing high shear and variable axial load. Some experimental programs have been started to gather the data necessary to formulate such a model. This model is needed to carry out realistic analysis of the actual performance of real reinforced concrete structures under seismic ground motion.

capacity of reinforced concrete joints. These problems will be discussed later.

Following is a summary of results from the above studies, and application of these results to seismic resistant design, starting with a suggested design criteria for the joints. Exterior joints are distinguished from interior joints. The summary is based on results and discussions in Refs. 25, 28, 39, 47, 119 and 124.

5.2.1 Design Criteria of Beam-Column Joints. Paulay in Ref. 47 suggested the following design criteria for joints in ductile moment-resisting space frames:

(1) The strength of a joint should not be less than the maximum strength of the weakest members it connects.

(2) The capacity of a column should not be jeopardized by possible strength degradation within the joint due to inelastic cyclic displacements of a frame.

(3) A joint should not be a prime source of energy dissipation.

(4) During moderate seismic disturbances a joint should respond within elastic limits so that no repair would be necessary for these inaccessible areas of the structure.

(5) The joint reinforcement that will ensure satisfactory performance should not present undue construction difficulties.

Although most researchers and designers agree with the above design criteria, approaches for practical design and detailing of joints vary considerably [25,28,47,116,118,124].

5.2.2 Exterior Beam-Column Joints. As Park [25] points out, an analysis of the forces acting on an external beam-column joint of a reinforced concrete frame (Fig. 20) and of the associated cracking shows that the bond conditions for the longitudinal beam and column bars are unfavorable because: (a) large steel forces need to be transferred to the concrete over relatively short lengths of bar; (b) flexural and diagonal tension cracks are present which will alternate in direction during cyclic loading; and (c) bond deterioration will occur during cyclic loading. For example, if the outer column bars are near to yielding in compression above the core and are yielding in tension below the core, approximately twice the yield force of the bar needs to be transferred to the joint core by bond over the depth of the core. The extremely high bond stresses so induced, and the anchorage forces from the beam bars, can result in vertical splitting of the concrete along the outer column bars. Thus the concrete cover over these bars spalls easily, particularly when heavy horizontal ties are used. This spalling may extend beyond the joint area, significantly reducing the flexural strength of the column, leading to hinging in the column rather than in the beam [42, 124]. Therefore, it has been suggested that the computation of column strength should be based on the strength of the column core area only [124].

If plastic hinging occurs in the beam at the column face, the anchorage of beam steel should be considered to commence within the joint core at one-half the column depth or ten bar diameters, whichever is less, from the face of the column where the steel enters. An anchorage block, in the form loading is that of gradually increasing the peak value of the load or deformation (Fig. 5[b]). This method can be conservative or not, depending on what element controls the behavior of the subassemblage. If the behavior is controlled by the beam or column, this loading sequence will give upper bounds for strength and energy absorbed and dissipated. If a lower bound is desired, it is best to use a sequence starting with large peak load and deformation cycles. However, if a weak panel zone controls the behavior, the gradually increasing load sequence will give a lower (conservative) bound. Another important consideration is the magnitude of peak deformations in each cycle and the number of cycles to which a specimen should be subjected. The magnitude of the peak deformation and number of cycles to which the specimen should be subjected depends on the type of construction as well as on the type of earthquake. Again, only integrated analytical and experimental studies can give correct answers.

(3) Overall behavior: Stiffness degradation observed with reversal of loading is considerably larger than that obtained for critical regions under pure flexure, or bending and low shear forces. The major factors contributing to this degradation for exterior beam-column connections appear to be: diagonal cracking in the joint; crushing of the concrete around the curved portion of the anchorage of the beam-column reinforcing bars; and grinding of the concrete in these regions and along the diagonal cracking, which increases with the number of cycles. No reliable method exists to predict the quantitative effect of these factors on the joint. Thus, there is a need for research on the behavior of joints under repeated reversal cycles. Behavior of interior beam-column connections also should be more thoroughly investigated than it has been to date.

(4) Seismic design: For exterior beam-column connections, premature failure of the joint can be avoided by beams or stubs framing into all four faces of this zone. If this is not possiblem it is advisable to: (1) use large numbers of small diameter bars for beam reinforcement rather than a small number of large-diameter bars; (2) use steel with a low yielding strength and a large plastic plateau or low strain-hardening modulus of elasticity; (3) use the widest possible column to increase length of anchorage, or extend the anchorage of beam bars into a concrete stub added in the outer face of the column; (4) design the shear reinforcement of the panel zone, neglecting the concrete's contribution in resisting shear and considering the maximum actual stress that can be developed in the reinforcing bars, including strain-hardening characteristics.

Some of these observations are still valid today, and some of the problems still remain, although beam-column joints in planar frames have been studied in many countries since 1972. Experimental results up to 1977 [25,28, 39,42,47,84,85,109-116] and their implications have been discussed by Park [25], Jirsa [28], Paulay [47], and Ohmori [84] during the workshop held at Berkeley [17]. The results of these studies have been incorporated in a series of recommendations [15,16] and even in new seismic code provisions [12, 14]. Although some of these recommendations have been questioned [117,118], there is no doubt that overall they are a step toward more efficient seismic resistant joint design.

Since 1977 new studies have been conducted on beam-column joints; some of which are reported in Refs. 29, 43, and 119-124. However, all these studies have been concerned with joint strength. There has been very little improvement in predicting stiffness, stiffness deterioration, and deformation subassemblage remained fixed in position, the other three hinges could be displaced horizontally upon application of a horizontal force at the lower hinge. At large displacements of the lower hinge, the P-δ effect caused by the vertical load in the column was significant.

Eight similar subassemblages have been tested to date. A brief discussion of the major results follows. One of the cantilever specimens was tested under a monotonically increasing load. The lateral load-deformation relationship (H vs δ) is shown in Fig. 21(c). From this figure, it can be seen that the curve is of the softening rather than the strain-hardening type. This is as to be expected from the results obtained with the beams, Fig. 21(b) together with the added P-δ effect. The significance of the P-δ can be noted from the comparison of the two curves shown in Fig. 21(c). Besides the H-curve, there is another one for the equivalent story shear, H_{eq}, which is related to the measured story shear by the relationship, $H_{eq} = H + P\delta/h_{col}$.

In Fig. 21(d), an analytic hysteretic loop is compared with the experimental one of Fig. 21(c). The agreement for the monotonically increasing story shear is excellent. However, large discrepancies can be noted during the loading in the reverse sense and these discrepancies become greatly magnified during the initial reloading of the second cycle. The following questions therefore arise:

(1) Why is there such a sharp degradation in strength during the first reversal, after just the first loading to a displacement ductility ratio of 4.5?

(2) Why is there such a pronounced degradation in stiffness during the first reloading, after just one cycle of a full reversal?

Since nominal shear stress developed in the beams was small [on the order of $3\sqrt{f_c' \text{ (psi)}}(0.25\sqrt{f_c' \text{ (MPa)}})$], similar to that induced in the cantilever beams of Fig. 21(b), it is clear that the observed degradation was not the result of shear in the beams. The main reason for this behavior was the slippage (pull-out) of the beams' main longitudinal reinforcement along the column joint. This is clearly shown in Fig. 21(e) where the sum of the measured pull-out and push-in of the steel bars is plotted.

The effect of repeated load reversals can be seen from the results presented in Fig. 21(f). These results were obtained from tests conducted on the specimen used in obtaining the results of Fig. 21(c) after it was repaired by injecting epoxy into the cracks. Although it was possible to achieve the strength attained during the first loading of the virgin specimen, this strength was achieved at a considerably greater deformation. During the second cycle, there was a large drop in strength from the first peak deformation reached during initial loading. As the number of cycles increased, both resistance and stiffness dropped as a result of bond deterioration along the embedment length of the beam bars.

Recently, there have been many studies of the interior joint [25,47, 114,117-123]. Many of the points made regarding exterior beam-columns apply to interior beam-column joints. In discussing ways to improve seismic behavior of interior joints, Park [25] points out that:

(1) When plastic hinging occurs in the beams at the column faces, it is recommended that the maximum diameter of longitudinal beam reinforcing

of a beam stub at the far face of the column where the longitudinal beam bars can be anchored (Fig. 20[c]) has been shown to improve joint performance and is being used by some designers in New Zealand. The maximum diameter of longitudinal column bars should not exceed 1/20th of the beam depth for steel with f_y = 40 ksi = 275 MPa or 1/25 of the beam depth for steel with f_y = 55 ksi = 380 MPa.

It is recommended that the nominal shear stress, v_c, carried by the concrete shear resisting mechanisms in the joint core should only be taken into account if the compressive load on the column exceeds $0.1f'_c\ A_g$. The degradation of shear carried by the concrete occurs due to repeated opening and closing of diagonal tension cracks in alternating directions and full depth cracks in the beam which results in the beam compression being transferred into the joint core by bond. The ACI 318-71 [35] assumption at 45° cracking is difficult to justify since the cracking will be parallel to the diagonal compression strut which runs from corner to corner. Hence, the design horizontal shear force in Fig. 20(a) is T-V', where T is the force in the beam bars enhanced for overstrength and V' is the column shear force. This design shear force should be resisted by the concrete, if the compressive load exceeds $0.1f'_c$, and by the force in the horizontal shear reinforcement which crosses the corner to corner crack. Vertical shear reinforcement should also exist in the form of vertical column bars around the perimeter of the column section (spacing not to exceed 6 in. (150 mm), with at least one intermediate bar between the corners. Such vertical bars are necessary to help transfer vertical shear forces. That is, four bar columns should not be used. A procedure for the design of vertical shear reinforcement has been developed [125].

The use of all these rules could lead to very conservative joint construction, but until new data is available, such requirements should not be relaxed.

5.2.3 Interior-Beam-Column Joints. Until 1972, relatively little attention was paid to interior-beam-column joints. This could have been due to the philosophy of some seismic codes regarding anchorage of the beam bars in this joint. For example, the commentary of ACI 318-71 and even ACI 318-77 [35] states, "The code does not require anchorage calculations for top and bottom reinforcement continuous through beam-column connections except for anchorage within each flexural member". The argument given is that "reverse loading tests of interior connections conforming to ACI 318-71 provisions show that the advantages of continuity offset any theoretical deficiencies in embedment length within the connections". Bertero and Popov, in Ref. 39, have questioned the soundness of this provision, because the slippage of the longitudinal beam reinforcing bars through the joint can lead to deterioration of the subassemblage's energy dissipation capacity. The importance of this degradation is illustrated in Fig. 21, which shows test results for one specimen [39,41].

Using the third-floor framing in a 20-story moment-resisting reinforced concrete building as a prototype, a half-scale subassemblage with an interior joint was designed (Fig. 21[a]). In this subassemblage, inelastic action was to develop in the beams, i.e., the design philosophy of strong columns-weak girders was followed. The beams were reinforced in exactly the same manner as beam specimens of a half-scale cantilever series of experiments (Fig. 21[b]) [40,69]. The testing arrangement for the cross-shaped specimen was such that an axial column force, as well as vertical forces at the ends of the beams, could be applied to it. Whereas the top hinge of the in beams, to reduce the danger of slippage, may result in the use of an excessive number of bars. Some designers have found it necessary to increase member sizes for the sake of steel placement within the joint. In spite of these measures, in conventionally reinforced joints a satisfactory safeguard does not yet appear to exist against pullout of beam bars from joints. Whenever practical, the prime cause of these difficulties, beam hinges adjacent to column faces, should be eliminated. This may be achieved by curtailing the beam reinforcement so that a deliverate weakness in flexural resistance results at a more suitable beam section. The relocated potential plastic hinge should be as near as practicable to the column face but far enough to ensure that, as a consequence of reversed cyclic loading, yield penetration will not extend to the column face. In such a beam when well designed, the steel stresses at the column face will approach but not exceed the level of nominal yield when the overstrength capacity at the relocated plastic hinges is simultaneously being developed. Therefore, if the joint core is adequately reinforced to resist horizontal and vertical joint shear force, it will remain elastic during cycling loading. This design philosophy, of moving the formation of plastic hinges from the face of the column and thereby assuring elastic joint behavior, was suggested by Bertero and Popov [38,39,68]. Experimental studies [42,119] show this to be a sound and practically feasible philosophy. Figures 23(a) and 23(b) illustrate one of the techniques used to move the beam inelastic regions (plastic hinges) away from the face of the column. (The specimen used is similar to that shown in Fig. 21[a].) The two top interior main bars of the beams were bent downward; and the two corresponding bottom bars were bent upward, intersecting 16 in. (406 mm) away from the column face. The hysteretic behavior of the specimen was excellent (see Fig. 23[c]). The hysteretic loops became pinched only after the first cycle with a full deformation reversal at displacement ductility seven. Comparison of test results of Figs. 23(c) and 21(f) shows a significant improvement achieved by moving the plastic hinge away from the column faces.

The above results have been confirmed by an experimental study carried out by Bull [126], and has been discussed by Paulay [127]. Paulay has also made recommendations which have been incorporated in the seismic provisions of the Draft New Zealand Code [14].

5.2.5 Prediction of Stiffness and Energy Dissipation Capacity of Beam-Column Joints. Analysis of results from investigations into the seismic hysteretic behavior of beams and beam-column subassemblages indicate that joints of R/C frames should not be considered rigid as is usually assumed. Two possible sources of deformation that may develop at the joint must be included to accurately predict the actual hysteretic behavior of the frame, particularly when large displacement ductility demands are expected. These two sources of deformation are illustrated in Fig. 24, and will be identified as the shear distortion of the joint ,γ_j, and the fixed-end rotation at the column face, θ_{FE}. Often the most important deformation is the one due to θ_{FE}. In contrast with the amount of research carried out to improve the design of beam-column joints for shear strength, very little has been conducted to improve methods of predicting stiffness, deformation capacity, and energy dissipation capacity of these joints. These mechanical characteristics are controlled by the θ_{FE}, which in turn depends on the bond-slippage characteristics of the beam bars along its embedment length at the joint.

Although excellent work has been done by several investigators on bond under generalized loading [128], to the best of the author's knowledge none of these investigations specifically addressed the problem of bond deterioration bars should not exceed 1/25th of the column depth for steel with f_y = 40 ksi = 275 MPa or 1/35th of the column depth for steel with f_y = 55 ksi = 380 MPa. The diameters of longitudinal column bars are limited as for exterior joints.

(2) The degradation of shear strength with cyclic loading occurs in the joint core for the same reason as in exterior joints. Repeated opening and closing of diagonal tension cracks, and full depth cracking in the beam at the column face, lead to a reduction in the effectiveness of the concrete diagonal compressive strut. Figure 22 illustrates the forces acting on a beam-column joint core. The forces entering the joint core are transferred across it by the diagonal compression strut (Fig. 22[b]) and by a truss mechanism involving diagonal tension and compression induced by the bond forces of the longitudinal bars (Fig. 22[c]). The shear carried by the concrete, v_c, arises mainly from the diagonal compression strut. When full depth cracking of the beam leaves the longitudinal steel as the only effective beam force transmitter, the mechanism involving truss action becomes dominant and this mechanism requires the presence of both horizontal and vertical bars to carry the diagonal tension forces across the joint core. Hence the force to be carried by the horizontal shear reinforcement increases as cyclic loading proceeds and vertical steel crossing the joint core is needed to carry the vertical forces necessary to complete the truss mechanism.

As noted by Paulay [47] although it is possible to transfer joint shear across the joint core with sufficient ties and intermediate vertical column bars, providing adequate anchorage for the main beam reinforcement presents a more difficult problem. The bond of the main beam reinforcement, anchored in the joint in the plane of the frame, can be adversely affected by the same mechanisms that are responsible for joint-core shear strength degradation: In particular by the transverse tensile strains imposed by the main reinforcement of the beams framing at right angles to the plane of the frame, and yield penetration into the joint when the inelastic regions (plastic hinges) developed adjacent to the faces of the joint. Generally, ACI 318-71 [35] development requirements cannot be satisfied for beam bars passing continuously through interior joints that are subjected to severe earthquake loading.

Excellent response to reversed cyclic loading (elimination of hysteretic pinching) was obtained at the University of Auckland [114] in specimens in which the steel forces were transferred to the core by welded bond (bearing) plates. Although this arrangement cannot be considered as a practical solution to the joint problem, the tests have clearly shown the great significance of proper anchorage within the joint.

When plastic hinges may form adjacent to columns, the diameter of the steel beam bars, passing through a joint, should not exceed the limits indicated above: 1/25th or 1/35th (depending on the grade of the steel) of the column depth in the relevant direction. If this is done, experimental evidence indicates that a large number of excursions with adequate ductility in both directions of seismic loading can be made before slippage of the bars will reduce the strength of the joint [47].

5.2.4 Elastic Joints. Two of the critical aspects of joint seismic behavior discussed above have been found to result in construction difficulties [47]. Unless the flexural tension reinforcement content in beams is kept small (i.e. less than 1.5 percent) the horizontal joint stirrup reinforcement may become so large that serious congestion of steel results. The limitation of bar size framing into the connection. Even the new recommendations of the ACI-ASCE Committed 352 [116] for design of beam-column joints allows an increase in the shear stress carried by concrete when the joint is confined by lateral members framing into the joint. It is agreed that transverse confinement can enhance the shear capacity of the concrete. However, the question is how effective this confinement can be when critical regions (plastic hinges) are developed in the beams framing transversely into the joint.

In Ref. 25 Park has shown that if the beams in the two directions are identical and they yield simultaneously, the horizontal shear force acting along the diagonal of the joint cross section (Fig. 25) is $\sqrt{2}$ times the uniaxial shear force. However, the diagonal tension crack intersects the same number of reinforcing bars as for uniaxial shear. If these bars are parallel to the sides of the section, the diagonal component of the bar force is only $1/\sqrt{2}$ that available to resist uniaxial shear. Hence design for biaxial shear for symmetrical two-way frames can lead to approximately double the quantity of shear reinforcement required for uniaxial shear design. This can create serious practical problems, such as congestion of steel. Experimental studies of this problem are needed. Some experiments are presently being carried out at the University of Canterbury, New Zealand [121], and at the University of Texas, Austin, Texas.

5.4 Concluding Remarks

Research concluded since 1972 has resulted in significant advance in understanding the behavior of beam-column joints, leading to development in the practical design and construction of such joints. However there are some problems that still need further research and development. There is a need to study how the strength capacity of the joint can be affected by (a) the slab; (b) 3D loading; (c) the eccentricities of the elements framing into the joint; (d) the amount and type of both transverse and longitudinal reinforcement. The main parameters controlling such strength capacity should be identified. There is also an urgent need to study the joint's stiffness, the deterioration of this stiffness, and its deformation capacity and energy dissipation capacity. It is important to develop simple but reliable mathematical models of joint behavior that can be used in computational analysis to study the affect of joint behavior on seismic response of ductile moment-resisting space frames.

Until further information is available, joint design should be based on the stringent rules given above or should be based on the philosophy of keeping the joint elastic by moving potential critical regions in the beams away from the face of the columns.

developing at the joint of an interior column. In the case of a joint in an interior column, we are dealing with bond-slippage of steel bars which are embedded in a well confined reinforced concrete but which can still be adversely affected by the mechanisms discussed in section 5.2.3. At Berkeley, there has been an investigation of the simplified problem of bond-slippage of bars embedded in well confined reinforced concrete, which simulates the conditions of a beam-column joint in a plane frame loaded laterally in its plane [129-133]. From the results of these experimental and analytical studies it has been concluded that:

(1) The assumption that beam-column joints of moment-resisting R/C frames are rigid needs to be reexamined. The main reinforcing bars of the beams do pull-out, and thereby cause beams to experience fixed-end rotation. The consequences of this behavior on the overall structural response must be examined.

(2) In the joints, it is essential to distinguish between the bond of unconfined concrete in the column cover from that of the confined core. The latter is appreciably better.

(3) Under monotonically increasing loads, when the beam main bar reaches yielding the accompanying pull-out can cause a fixed-end rotation in the order of 0.001 radians.

(4) The displacement of a bar due to monotonic loading at the column face can be estimated using simple idealizations of bond stress distribution [131]. The dependence on concrete strength, type of lugs, embedment length, concrete confinement, etc. requires further investigation.

(5) Significant bond deterioration occurs from cyclically applied load reversals, particularly when the applied stresses exceed yield.

(6) It appears that bond resistance deterioration is gradually stabilized at the value of friction between two concrete cylindrical surfaces which have a common diameter equal to the outer dimension of the bar, including the lugs.

(7) More comprehensive analytical models are required for generalized loading of a bar. (A model has been developed by Viwathanatepa [133].)

(8) The implications of the effect of θ_{FE} on the behavior of structural systems should be studied analytically. (A computer program that permits inclusion of θ_{FE} in nonlinear analysis has been developed by Soleimani [120].)

5.3 Beam-Column Joints of Space Frames Subjected to 3D Loading

As pointed out in the discussion of columns under 3D loading, the moment-resisting frame is usually a space frame having two-way frames in each joint, i.e., beams framing into the joint along the two orthogonal main axes of the structures, and subjected to ground motions with components in both directions. In spite of this, most seismic codes presently require that the joint be designed independently in each direction. Furthermore, some codes, such as ACI [35] allow the transverse reinforcement in the connection to be reduced by one-half if every beam has a width not less than one-half the column width and a depth not less than three-fourths that of the deepest beam the confinement pressure, f_r.

The maximum compressive strength $f_c^* \max$, occurs after some strain, ε_0^*, and can be related to the unconfined compressive strength of the same concrete, f_c, and the confinement pressure as:

$$f_c^* \max = f_c + k_o f_r \qquad (3)$$

At very large deformations, $\varepsilon_u^* >> \varepsilon_0^*$, the compressive strength usually decreases to a value of f_{cu}^*, and can be related to these same parameters as:

$$f_{cu}^* = f_c + k_u f_r \qquad (4)$$

The confinement pressure, f_r, depends on the geometric and material characteristics of the spiral wire, and can be approximated by:

$$f_r = \frac{2A_{sp}\ f_s}{D_c \quad s} = \frac{1}{2} \rho_s\ f_s \qquad (5)$$

where ρ_s is the ratio of volume of spiral to total volume of core and f_s is the stress that had been developed by the spiral wire. Assuming that the ductile spiral wire yields when the longitudinal strain in the concrete is in the range ε_0^* to ε_u^*, and that the strain-hardening of the spiral is negligible in the range of these concrete strains: (a) f_s is equal to f_y; (b) f_r can be calculated for given values of A_{sp}, D_c, and s from Eq. 5; (c) values of k_o and k_u can be calculated from Eqs. 3 and 4, using the test results. The values for the five different concretes used in this study are shown in Table 1. Early investigators have shown that the confinement effectiveness coefficient, k, varies with lateral pressure intensity and with longitudinal strain. However, in developing the ACI criterion for spiral reinforcement (Section 10.9.2 of ACI 318-71) [35] and similar criteria which are based on the confinement of concrete, a constant value of k, usually taken as 4.0 to 4.1, has been assumed.

TABLE 1.- EFFECT OF CONFINEMENT ON COMPRESSIVE STRENGTH AND DEFORMATION OF CONCRETE.

Type of Concrete	Confinement Stress Ratio	Maximum Compression		Ultimate Compression	
		Strain Ratio	Confinement Effectiveness	Strain Ratio	Confinement Effectiveness
	(f_r/f'_c)	($\varepsilon_o^*/\varepsilon_o$)	k_o	($\varepsilon_u^*/\varepsilon_o$)	k_u
Normal	0.13	2.8	7.0	11.5	0
E-5	0.32	7.8	5.0	11.5	3.1
Lightweight	0.13	1.9	4.4	8.7	-0.5
R-5	0.32	4.0	2.0	6.7	2.0
B-5	0.13	1.35	3.9	10.6	0
	0.32	1.85	1.0	8.6	0.9
R-3	0.11	1.8	2.7	8.9	-1.0
	0.24	5.9	2.5	8.9	2.0
B-3	0.11	1.7	1.35	11.6	0
	0.24	8.0	2.1	9.0	2.1

6. SEISMIC BEHAVIOR OF LIGHTWEIGHT CONCRETE LINEAR STRUCTURAL ELEMENTS AND THEIR CONNECTIONS

6.1 General

There are a number of advantages to using lightweight, rather than normal weight, aggregate concrete in seismic-resistant reinforced concrete construction. One of the basic principles of such construction is to avoid use of unnecessary mass. The lower the weight of the reactive masses the lower the seismic forces that will develop as a consequence of earthquake ground motions. If one compares the standard mechanical characteristics obtained from compression test per unit weight of lightweight concrete with those of normal weight concrete (Fig. 4) or analyzes results available from experimental studies on individual structural elements there is no doubt that it would be advantageous to use lightweight aggregate concrete. Therefore, some investigators have concluded that the use of this type of concrete results in more efficient earthquake resistant construction [134, 135]. However, proper assessment of the performance of any structural system requires not only analysis of the behavior of the individual elements, but also of the assemblage of these elements. As already discussed, this is of particular importance in the case of R/C structures where connections between elements depend upon transfer of forces between the two constituent materials, reinforcing steel bars and concrete.

Current seismic codes in both the U.S. [35, 36] and Canada [136] permit the use of lightweight concrete in the construction of ductile moment-resisting space frames. The only precaution is that "the maximum specified strength for lightweight concrete shall be limited to 4000 psi (28 MPa)". Unfortunately, because of its lower modulus of elasticity, very high compressive strength concrete mixes have been used to achieve a higher degree of stiffness and this has caused some problems regarding the use of these mixes for seismic-resistant construction, particularly regarding the effectiveness of confinement, bond, and shear transfer of such concrete.

6.1.1 Confinement. References 20, 23 and 137 discuss the problems of using confined lightweight aggregate concrete for seismic construction. A summary of the observations made in these references follows.

Confinement of concrete with all types of aggregate tested was effective in developing large deformability. However, the effectiveness of concrete confinement in the performance of earthquake-resistant reinforced concrete structures should not be based only on the extent to which the deformability is increased, but also on the ability of the confined concrete to sustain large deformations without loss of strength. Therefore, confinement should also increase the compressive strength of the concrete, so that it is possible to offset the loss of strength due to the reduction of the cross-section resulting from crushing and spalling of the concrete cover.

Figure 26 shows some results of the study in Ref. 23. These results show that the conditions of increased deformability and compressive strength are satisfied to a verying extent for different concretes, and the effectiveness of confinement is highly sensitive to the type of aggregate used. The effectiveness of confinement can be characterized by two material constants, k_o and k_u, which are defined by relating the increased compressive strength f_c^*, to earthquake-resistant reinforced concrete structures rely on the beneficial effects of confinement on concrete behavior. Thus it is important to analyze the implications of the results summarized above [23] with regard to seismic behavior of concrete structures. Some observations obtained from such analyses follow [137].

1. Confinement of concrete with all types of aggregates is effective in developing large deformability, i.e. large ultimate strains. This characteristic is the major factor in the improved performance of elements with spirally confined concrete, as it compensates for some of the losses in strength and stiffness of concrete under cyclic loading.

2. The increase in compressive strength due to confinement is about twice as great for normal weight concrete as for lightweight concrete. Therefore, one should be cautious in using equations from tests on normal weight aggregate concrete to predict behavior in lightweight concrete.

3. The low effectiveness of confinement in some concretes may lead to significant losses in compression capacity when spalling occurs. This is of utmost importance in the seismic design of column elements, since these elements should be able at all times to resist the effects of gravity loads and overturning moments.

These conclusions have been confirmed in a recent experimental study[138].

6.1.2 The Bond and Shear Transfer Problems. Recent bond tests performed at Berkeley [129-133], on specimens simulating the conditions of an interior beam-column joint, demonstrated that the deterioration of bond in lightweight concrete occurred under smaller steel strains than in normal weight concrete [132].

In the case of flexural critical regions under high shear, one of the main factors controlling the degradation of stiffness is shear transfer along the cracks. Mattock has conducted a series of studies on the problem of shear transfer along cracked concrete[78, 79, 139]. Based on test data obtained in these studies, Mattock has concluded that "the shear transfer behavior of initially cracked all lightweight concrete is more brittle than that of sanded lightweight or sand and gravel concrete," and that "shear transfer behavior across a crack becomes more brittle as the concrete strength increases".

The above studies examined the three basic problems in the behavior of lightweight concrete - the effectiveness of confinement, bond, and shear transfer. The studies showed that, for seismic-resistant construction, lightweight concrete has certain deficiencies in addition to its low modulus of elasticity. These deficiencies indicate a need for further studies in order to properly modify the proportioning and detailing rules obtained from and used for members cast of normal weight concrete, so that these rules can be applied to lightweight concrete.

6.2 Behavior of Linear Elements and their Subassemblages.

6.2.1 Studies of Beam Behavior. Very few studies have been reported on lightweight concrete beams subjected to seismic action. Mihai, et al. [140] have

As shown in Table 1, the values of k for normal weight aggregate concrete vary in the range of 0 to 7.0. For the two lateral pressures (0.13 f_c and 0.32 f_c), values of k_o at maximum compression are 7.0 and 5.0 respectively, and values of k_u at ultimate strength are 0 and 3.1 respectively. Based on these values, and noting from Fig. 26 that concrete behaves in a relatively ductile manner throughout a significant range of strains, a constant value of k = 4.0 may be justified for normal weight concretes such as E-5, particularly in the case of $f_r = 0.32(f_c)_{10}$.

For lightweight concretes B-3, B-5, R-3, and R-5, the values of k vary in the range of -1.0 to 4.4. Negative values of k_u indicate that compressive failure in the confined concrete may occur at values below the compressive strength of unconfined concrete. For the two lateral pressures ($f_r \approx 0.1\ f'_c$ and $f_r \approx 0.3\ f'_c$), values for k_o at maximum compression range from 1.0 to 4.4 and values for k_u at ultimate range from -1.0 to 2.1. Based on these results, a value of k in the range of 1.0 to 2.0 should be taken in developing design criteria based on the increase in strength due to confinement of lightweight concrete. Therefore, the amount of spiral steel required in a column of lightweight aggregate concrete will be 2 to 4 times greater than that currently prescribed by the ACI Code [35]. Because of the geometric limitations introduced by the size of the spiral wire and the minimum spacing, it would be virtually impossible to produce a spiral which would also allow normal placing of concrete.

The effect of the variable coefficient, k, is illustrated in Fig. 27. In this figure, the loss of the axial load carrying capacity for spirally reinforced concrete columns due to spalling is plotted against k, assuming that the spiral reinforcement was designed in accordance with the ACI criterion [35]. This loss of capacity is expressed as a ratio and derived as:

$$\text{Loss} = 0.85f'_c(A_g - A_c) - kf_rA_c \ ;$$

and using Eq. 5

$$\text{Loss} = 0.85f'_c(A_g - A_c) - 0.5k\rho_s f_s A_c \tag{6}$$

According to the ACI criterion, $\rho_s = 0.425\ [(A_g/A_c) - 1](f'_c/f_s)$. By substituting this equation into the above, and dividing by $0.85f'_cA_g$, the following ratio is obtained

$$\frac{\text{Loss}}{0.85f'_c\ A_g} = (1 - \frac{A_c}{A_g}) - 0.25k(1 - \frac{A_c}{A_g}) \tag{7}$$

Typical values of A_c/A_g (where A_c is the area of core and A_g is the gross area) for spirally reinforced square columns vary from approximately 0.4 to 0.6. For round columns this ratio varies from approximately 0.5 to 0.7. The loss ratio for typical values of A_c/A_g is plotted in Fig. 27, which illustrates the significant losses that can occur due to k values lower than 4.

Most of the recent suggestions and requirements for improved design of

Monotonic Loading - From analysis of the curves shown in Fig. 28(a) it is clear that the overall behavior of the lightweight concrete was very similar to that of normal weight concrete. Furthermore, the contribution of the fixed end rotation θ_{FE}, due to slippage of the beam main bars along the joint, to the lateral displacement, δ, was approximately the same for BC4 and BC7. However, the initial stiffness, which is highly dependent on the material stiffness of the concrete, was 52 percent higher in BC4. This was in agreement with the relative moduli of elasticity of the two specimens, as BC4 had a 46 percent higher modulus of elasticity. This signifies that lightweight R/C structures will have greater nonstructural damage and higher P-δ moments for the same displacement ductility.

Cyclic Loading - The performance of the normal and lightweight concrete specimens under incrementally increasing cyclic loading differed significantly as shown in Fig. 28(b). Specimen BC3 reached a peak strength at LP25 ($\mu_\delta = 3.9$) and LP26 ($\mu_\delta = -4.2$) while the strength of specimen BC8 peaked much earlier; at LP17 ($\mu_\delta = 1.45$) and LP18 ($\mu_\delta = -1.75$). At LP22 ($\mu_\delta = -2.7$) the capacity of BC8 was already only 70 percent of that of BC3. The difference in behavior was due to the premature total slippage of the reinforcement in specimen BC8. By LP24 ($\mu_\delta = -2.7$), the contribution of the θ_{FE} at the column face to δ was over 75 percent for BC8 while it was less than 35 percent for BC3. Total slippage of the beam bars did not occur in BC3 until LP29 ($\mu_\delta = 5.4$) when over 50 percent of δ was due to θ_{FE}. This strikingly different behavior under cyclic loading indicated that the bond within the joint deteriorates at lower μ_δ in lightweight concrete. Although the cause of this earlier deterioration is not completely understood, it is speculated that the lightweight aggretate is sheared and crushed by the lugs of the deformed bars at lower stresses, leading to earlier bond deterioration. Propogation of cracks formed by the action of the lugs might also be affected by the type of aggregate used.

6.3 Concluding Remarks. From the available information, particularly from results of studies carried out at Berkeley, the following observations can be made. Because of the relatively meager data available, these observations are of a preliminary nature.

1. Individual lightweight aggregate members have a similar hysteretic behavior to normal weight aggregate members of similar strength. The only remarkable difference is the lower stiffness of lightweight concrete, which means larger deformation is needed to develop the same displacement ductility.

2. Beam-column subassemblages subjected to monotonic loading show that a displacement ductility (μ_δ) in excess of 5 can be achieved without a decrease in resistance. Behavior is very similar to that of the normal weight specimen. For the same ductility displacement ratio the total displacement and the story drift is greater than that of the normal weight specimen, causing larger P-δ moments.

3. Under cyclic loading, the behavior of beam-column subassemblages cast of lightweight aggregate concrete is drastically different than that under monotonic loading, due to earlier slippage of the beam reinforcement through the joint. Yielding of this reinforcement accelerates bond deterioration and therefore slippage.

4. Under cyclic loading, the energy dissipated by beam-column subassemblages is smaller than that of similar normal weight subassemblages. The main reason for this was that total slippage of the beam reinforcement through

carried out some tests on lightweight aggregate concrete beams columns and their connections and have concluded that:

"Generally the ductility of bending members of granulite lightweight concrete is 15-40% greater in comparison with that of similar members of heavy concrete. In the case of members subjected to compression with bending, the ductility factors are close for the similar members made of heavy concrete, and lightweight concrete. With a proper detailing conception, the joints realized with lightweight concrete are more ductile, with 15-25% increases, in comparison with heavy concrete ones. The more elastic and also more breakable behavior of lightweight concrete, requires detailed and careful experimental and theoretical research for all types of granulite material".

Because of insufficient detail it is difficult to judge what definition of ductility the author of Ref. 140 has used.

6.2.2 Studies of Column Behavior. Experimental studies show it is possible to achieve good ductile behavior by properly confining lightweight concrete with spiral or closely spaced and carefully detailed rectangular hoops and ties [140, 141, 142]. However the only comparison available between similar specimens cast of lightweight and normal aggregate concrete show better strength, stiffness and ductility for the normal aggregate concrete [142]. In Section 6.1, some drawbacks of the use of lightweight aggregate concrete were discussed. In addition, lightweight concrete has a higher rate of creep than normal weight concrete. Therefore, serious questions remain regarding the use of lightweight concrete in columns, especially in tall frame buildings. In the lower stories of buildings, high axial loads caused by gravity loads can cause : a higher rate of creep and larger P-δ effects of lightweight than for normal weight concrete, due to the lower stiffness of lightweight concrete. Comprehensive experiments are needed to find the role of these effects on the hysteretic behavior of lightweight concrete columns.

6.2.3 Subassemblage Behavior. As discussed in Section 6.1, proper assessment of the performance of any structural system requires studying the behavior of the system's basic subassemblages. Studies were conducted at Berkeley [29] of the behavior of basic subassemblages of a ductile moment-resistant space frame (DMRSF) built of lightweight aggregate concrete. The completed study had two main objectives. The first was to study the behavior of a DMRSF subassemblage constructed of lightweight aggregate concrete under earthquake-like load conditions and to compare such behavior to that observed under monotonic loading, paying particular attention to the effects of bond degradation in the joint region. The second objective was to compare the performance of lightweight R/C subassemblages to that of previously tested normal weight subassemblages for both monotonic and cyclic loadings. Figure 21(a) shows the specimens which were used: half-scale models of interior beam-column subassemblages from the third floor of a twenty-story office building. A summary of the results of these tests follows.

Figure 28 compares the behavior of lightweight aggregate specimens (BC7 and BC8) with that of normal weight subassemblages (BC3 and BC4) of similar concrete strength and steel yield strength subjected to similar applied displacement programs. Due to the greater flexibility of lightweight concrete, ductility, μ_δ, rather than absoluted displacement was used as the base of comparison.

7. SEISMIC BEHAVIOR OF PRESTRESSED AND PRECAST R/C LINEAR ELEMENTS AND THEIR CONNECTIONS

7.1 General

7.1.1 Prestressed Concrete. In 1972, the author reviewed the state-of-the-art in prestressed and partially prestressed concrete structures and their elements [16]. He reported the conclusions reached by Blakely and Park in their historical review of the seismic resistance of prestressed concrete (1971) [144], as well as the conclusions of their tests on four full-size, precase prestressed concrete beam-column assemblies. A brief summary of these conclusions follows:

From the 1971 review:

(1) Most structures containing prestressed concrete elements which have been subjected to earthquakes have performed well. Failures which have occurred appear to have been due mainly to failure of the supporting structures or of the joint connections. However, there is very little information on the behavior of fully framed prestressed concrete structures under strong earthquakes.

(2) Although the energy absorbed by a prestressed concrete member could be the same or even larger than a similar reinforced concrete member the greater elastic recovery of the prestressed concrete member will result in a lower energy dissipation for cyclic loading. This lower energy is a drawback in seismic design. However, little is known of the energy-dissipation capacity of prestressed concrete members under high-intensity cyclic loading.

(3) High intensity cyclic loading tests of prestressed concrete members and subassemblages including different joint details is needed.

From the test results:

(1) Energy-dissipation is relatively small prior to commencement of crushing in the concrete, but substantial once crushing has occurred. (2) Large post-elastic deformation can be available in prestressed concrete members, even where the transverse reinforcement satisfied only normal prestressed-concrete code requirements for shear. (3) Substantial stiffness degradation is apparent for prestressed concrete members after high-intensity cyclic loading. (4) Mortar joints between precast post-tensioned frame members can behave satisfactorily under high-intensity load reversal. (5) Prestressed-concrete framed structures can be capable of resisting moderate earthquakes without structure damage, and of withstanding severe earthquakes although structural damage may occur, with a consequent difficulty of repair back to fully prestressed condition.

In the concluding remarks of Ref. 16, the author enumerated a series of problem areas in which research was needed to improve understanding of the behavior of concrete structures under generalized excitations. The author then stated, "All the above required research applies as well to reinforced concrete as to prestressed concrete. However, in prestressed concrete other problems such as questions of the optimum degree of partial prestressing, of bonded versus unbonded prestressing tendons, the behavior of prestressed anchorage under dynamic loading, etc. still remain to be answered." The author would like to emphasize that the basic problems encountered in the seismic the joints occurred earlier in the lightweight specimen, at $\mu_\delta = 2.4$ as compared to $\mu_\delta = 5.4$ for the normal weight specimen, resulting in a more pinched hysteretic behavior.

5. The assumption of a rigid joint appears to be inaccurate not only at large ductilities, but even at the yield level, under monotonic and particularly cyclic loading. The contribution of the fixed-end rotation to the total story drift under monotonic loading is about 13 percent at the yield level, increasing to 22 percent at higher ductilities. Under cyclic loading the contribution is 18 percent at the yield and increases to greater than 90 per cent at higher ductilities.

6.4 Recommended Design Improvements and Research Needs.

1. Development of new design and construction methods is needed to prevent yielding of the reinforcement at the beam-column interface, which usually triggers or accelerates total slippage of the beam reinforcement. One such method is to move the regions of the inelastic action away from the joint. This can be accomplished by: (i) bending or cutting off at a short distance from the joint some of the top and bottom beam reinforcing bars, forming a region of sufficiently lower moment capacity to be the critical one. Some research has already been conducted in this area using normal aggregate [119, 126]; or by (ii) designing haunches which sufficiently increase the moment capacity near the joint to prevent yielding of beam reinforcement at the column face. Another method consists of improving the anchorage of the reinforcement within the joint by using special mechanical devices [114] or better detailing, such as crossing the top and bottom beam reinforcement [127,143].

2. The basic causes for more rapid bond deterioration in lightweight concretes should be explored further.

3. Experiments with beam-column subassemblages having floor slabs are needed to more accurately simulate the actual conditions found at joints in buildings.

4. Analytical programs need to be developed, based on a stiffness degradation model, which include fixed-end rotations at the joint in order to study the affect of the observed deterioration on the response of framed structures to earthquake ground motions.

7.2 Seismic Behavior of Prestressed Concrete Beams, Columns, and their Subassemblage

7.2.1 Beams. As with ordinarily reinforced concrete structures, it is convenient to classify prestressed beams according to stresses controlling behavior of their critical regions: i.e., Flexural and Flexural with High Shear.

7.2.1.1 Flexural Critical Regions. Hawkins [147], after analyzing the experimental results obtained in numerous experiments as well as the performance of prestressed concrete beams in real earthquakes, drew a series of conclusions. The most important conclusions are summarized below, together with some conclusions from recent studies carried out in New Zealand [148,152].

(1) Most prestressed concrete beams, when designed for loading reversals, perform well in earthquakes. Generally, deformed bar reinforcement and confinement by stirrups are necessary to provide adequate strength under moment reversals. The failures that have occurred have been due mainly to failures of the supporting structures or connections. Major consideration must be given to the strength of connections and supporting structures.

(2) Experimental flexural strengths of the beams are usually greater than theoretical flexural strengths because experimental moments reach their maximum at an extreme concrete fibre strain greater than 0.003. This is due to the extra confinement given to the beam concrete by its reinforcement and the adjacent column concrete. With stirrups and compression reinforcement, ultimate strength can increase by as much as 16 percent.

(3) Unless the first damaging load exceeds about 80 percent of the collapsed load, the capacity in the reverse direction is unaffected. If the concrete is not confined, cycling to strains greater than 0.002 induces a loss in strength and stiffness due to spalling of the compressed concrete and penetration of crushing into the core of the member. That degradation can be slowed and the ductility and energy absorption increased by the addition of either bonded compression reinforcement or confinement - preferably both. Unless confinement is provided there is a marked degradation in the flexural capacity for beams reversed cyclically and loaded to an excess of 90 percent of their flexural capacities. Confinement should be achieved by closed stirrups with a spacing not exceeding d/4.

(4) High seismic loading rates can result in strength increases of four to seven percent and ductility increases of 10 to 15 percent. It is generally appropriate for design computations to be based on static loading strengths only.

(5) Prior to crushing of the concrete or marked inelastic flow of the prestressing steel, loading-unloading curves are bilinear with ranges corresponding to crack open and crack closed conditions. The loading and unloading curves closely parallel each other with small amounts of dissipation of energy.

(6) Prestressed beams show marked elastic recoveries even after considerable inelastic deformations, leading to pinching of the hysteretic loops. Figure 29 compares beam moment-end deflection relationships for three beam-column specimens with similar theoretical flexural strengths and with prestress levels of 1160, 386 and 0 psi,(8.1, 2.7 and 0 MPa,) respectively. Energy dissipation for prestressed concrete elements is less than that for

behavior of ordinary reinforced concrete are also present in prestressed concrete, since prestressed concrete is just a special case of reinforced concrete structure in which an initial, desirable state of compression is introduced to the concrete. The only difference is the degree of severity of these problems. (For example, one cannot expect good seismic hysteretic behavior of prestressed elements whose critical regions have not been properly confined with lateral reinforcement. These points - that the basic problems of ordinary and prestressed concrete are the same, and that the severity of the problems may differ - should be kept in mind in juding results from experiments of prestressed concrete elements.

In order to obtain a good sense of the state-of-the-art and the state-of-the-practice, up to 1977, of seismic behavior of prestressed concrete framed structures and their elements, one can review papers presented at the ERCBC Workshop held at Berkeley in 1977 [17]. Particularly appropriate are the papers by Lin and associates [145]; Park [146]; Hawkins [147]; and Park and Thompson [148]. Hawkins, in Ref. 149, has reviewed and synthesized the information presented in this workshop and several other researchers and practicing engineers have discussed it. From this review it is apparent that although the advances in knowledge about seismic behavior of prestressed concrete elements have been not as great as the advances for ordinarily reinforced concrete elements, there is sufficient evidence to formulate comprehensive seismic design recommendations for prestressed concrete [150,151]. A brief summary of some of the new information on seismic behavior of prestressed concrete elements is presented later in this section.

It is generally agreed that the response of a prestressed concrete structure to a given earthquake will be greater than that of a comparable reinforced concrete structure, because of its lower energy dissipation and viscous damping properties. However, because the use of higher concrete strength results in a smaller neutral axis depth, prestressed concrete members may sustain greater curvatures before crushing than comparable reinforced concrete members of the same flexural strength and section size. Alternatively, prestressed concrete members may be of smaller section, and therefore less mass. These factors may well counteract the effect of the smaller energy dissipation capacity under cyclic loading [152]. From the review of all the available information, it becomes apparent that proper use of prestressing can be an asset to seismic resistant construction of concrete frame structures.

7.1.2 Precast Elements. In the zones of high seismic risk in the United States, precast concrete framing is not widely used as a primary lateral load resisting system: little information exists regarding seismic behavior of this type of concrete construction. Hawkins, in Refs. 147 and 149, reviews the state-of-the-art in seismic resistance of precase concrete structures, although most of the review is devoted to precast panel construction rather than to precast concrete frames. Ikeda and associates, in Ref. 153, have reviewed the state-of-the-art of precast concrete techniques in Japan, pointing out that the main problem is the prediction of strength and deformation capacities of beam-column connections. It is clear that there is nothing wrong with the elements. The problem is in the joints between these elements. It is believed that proper use of prestressing can improve the performance of joints between precast elements. There is a tremendous potential for the use of lightweight aggregate concrete, precast, prestressed elements in seismic resistant construction.

steel is necessary in prestressed columns (as it is for reinforced concrete columns) once the axial load exceeds some nominal value such as 0.1 P_0 where: P_0 = strength of columns when load is applied with zero eccentricity.

Prestressing can improve the behavior of reinforced concrete columns [142] and therefore of the whole frame, provided the peculiarities of prestressing are considered in the design as well as in the detailing of the columns. Figure 30 illustrates an example of post-tension prestressing the outer columns of the first 5 stories of an 18 story building (to reduce the possibility of tensile cracking strength during severe earthquakes). This application has been discussed by Ohmori [84], Muto [94] and Hisada and associates [142].

There is a need for experimental work on partially prestressed columns under severe seismic actions. Among the parameters that need to be studied the following deserves special attention:

- optimum degree of prestressing, and optimum location of the pressure line;

- quantity of confining steel necessary to achieve adequate rotation ductility, particularly under high compressive loads, and to prevent buckling of the bars;

- the affects of unbonded tendons, particularly when used continuously over several column stories.

7.2.3 Beam-Column Joints: Following design criteria similar to that used for ordinarily reinforced concrete structures, the FIP Commission on seismic structures [150] recommends: "The connections between members in prestressed concrete construction should be carefully designed for effectiveness at all earthquake limit states, on the following basis:

(a) Connections should be checked for both seismic stresses and deformations.

(b) The load-carrying capacities of connections should not be less than those of the adjacent structural members.

(c) Connections should be capable of failing in a ductile manner. "

In their commentary the FIP Commission emphasizes that inelastic loading cycles (particularly those involving not only load but also deformation reversals) can result in a degradation of the concrete shear-resisting mechanism due to breakdown of the joint core, caused by alternating bond force and diagonal tension cracking.

The above design philosophy is clear and well accepted. However, adequate provisions, methods and rules for quantifying and practically applying this philosophy are still lacking despite improvements in understanding hysteretic behavior of beam-column joints. Most of the studies have been related to the strength of the joint, very little has been done regarding prediction of stiffness and its degradation with increasingly severe cyclic loading, or with the prediction of deformation capacity and energy dissipation capacities.

reinforced concrete elements because of elastic recovery effects. In general, the residual tensile force in the prestressing steel is adequate to close previously open cracks. Thus, significant energy dissipation does not develop until the deformed bar reinforcement yields, the prestressing steel yields, or the concrete crushes. Recent test results of beams where flexural behavior controls inelastic response have been promising. The use of these beams in seismic resistant prestressed concrete frames should be investigated further. Most previous tests have primarily involved symmetrical arrangements of prestressed and nonprestressed steel: these tests need to be extended to other arrangements. Further study is also needed of the spacing of stirrup ties that are required to prevent buckling of nonprestressed steel under reversed loading.

7.2.1.2 Flexural Critical Regions with High Shear. As noted by Hawkin's [147], there is little information available on the behavior of these prestressed critical regions. In the tests carried out by Park and his associates [148, 152], the nominal unit shear stressed developed were very small: less than $2\sqrt{f'_c}$ (psi) [$0.16\sqrt{f'_c}$ (MPa)] and less than 1/3 of the theoretical computed shear strength using ACI 318-71 [35]. Therefore, no adverse shear effects were observed. There is an urgent need for systematic studies on the behavior of the prestressed elements subjected to high shear stresses.

There is general agreement that beams should be proportioned and detailed so that they will not fail in shear. The FIP Commission [150] recommends that in calculating the design shear force the plastic hinge moments should be determined considering the possible overstrengths of the material. These enhanced plastic hinge moments may be estimated as 1.15 times the flexural capacities based on the characteristic strengths of the materials. The proposed provisions for the New Zealand Code [14] contains specific requirements for designing against shear force, neglecting the concrete's contribution in resisting shear when the design axial compressive force produces an average stress smaller than $0.1\ f'_c$.

7.2.1.3 Bond, Grouting and Anchorage. According to Lin and associates [145], seismic safety can be equally obtained by either bonded or unbounded construction. However, this is a controversial issue, on which the FIP Commission on Seismic Structures has prepared a special report [145]. Present FIP guidelines [150] recommend grouting the prestressing ducts in flexural members of a ductile structural frame. The New Zealand Code has similar requirements, except for special cases where post-tensional tendons may be ungrouted. Bond transfer lengths and performance under cyclic loading are very sensitive to surface conditons and to the method of release for the strand.

Careful consideration must be given to the location of tendon anchorages. They should not be placed in regions of high bending or rotation, which can adversely affect their capacity. Consideration must also be given to the flow of forces from the anchorage.

7.2.2 Columns. Except for experiments carried out by Hisada and associates [142] there has not been much research done on prestressed columns alone. Usually, columns have been studied as part of a subassemblage, in which they were stronger than the beams and hence were not critical elements. An exception to this was the joint core regions which will be discussed later. The ductility of prestressed concrete columns have been studied by Blakeley [154]. As expected, the available curvature ductility of a prestressed concrete column decreases with increased axial load level. Special transverse confining and fabricators of these precast elements try to locate the connections so they can be easily constructed and are not subjected to severe simultaneous bending, shear and axial forces. An example of proper location of field connections is shown in Fig. 32.

7.4 Concluding Remarks

Prestressed and precast linear concrete elements are not widely used to form primary seismic resistant structural systems. The amount of research in this area has been relatively small compared with that on ordinarily reinforced concrete, and some fundamental questions remain unanswered. Nonetheless, in the last decade, there have been significant advances in understanding problems introduced by these techniques of reinforced concrete construction.

There is tremendous potential in the use of prestressed and precast lightweight concrete structural elements. To realize this potential quickly, it is necessary to recognize - that prestressed and precast concrete elements are just a particular case of R/C structures and practically all drawbacks of ordinary R/C elements are also present in prestressed and precast elements. Therefore, existing knowledge of seismic behavior of the ordinary R/C elements should be used. The problems to concentrate on are those that are peculiar to prestressing (i.e. problems of anchorage, bond, transfer, grouting, type of steel and level of prestressing); and to the precasting technique (like the problems of joint).

The work of Park and his associates [146,152] has significantly increased knowledge of the effects of prestressing on joint behavior. Their work showed that serious difficulty in preventing joint core distress during severe seismic loading can only be minimized by careful proportioning and detailing. Their main findings follow:

(1) The ACI 318-71 Appendix A [36] approach for joint core shear strength cannot be regarded as adequate for plane frames subjected to intense cycles of seismic loading. It fails to make any provision for vertical shear reinforcement in the plane of bending.

(2) The use of a reasonable level of prestress through a central tendon improved the hysteretic behavior of the joints.

(3) The contribution of the concrete to shear strength should be neglected except when the mean column compressive stress exceeds to 0.1 f'_c.

(4) The inclusion of vertical shear reinforcement within beam-column joint cores, in the form on intermediate column bars, and horizontal shear reinforcement, in the form of ties, allows the joint core shear force to be resisted more effectively than when intermediate column bars are not present (Fig. 31).

(5) The draft of the New Zealand Concrete Design Code [14] recommends the provision of the vertical shear reinforcement to transmit vertical shear forces within the joint core. The amount of horizontal and vertical shear reinforcement required by this draft Code approach was found to be safe but rather conservative.

Although the above results led to improved understanding of the hysteretic behavior of prestressed concrete beam column joints, research is needed in the following areas:

(1) The actual contribution of concrete to joint strength, stiffness and energy dissipation capacity when subjected to different levels of compressive stress.

(2) Other means of vertical joint shear reinforcement.

(3) Maximum bar diameters allowable for longitudinal steel to prevent total slippage through the joint core.

(4) The affect of unbounded tendons.

(5) The potentials of moving the critical regions away from the face of the columns.

7.3 Seismic Behavior Precast Concrete Beam, Column and their Connections

As discussed in Section 7.1.2, the main problem in using these elements is associated with their connections. As noted by Hawkins [147], while many types of connections have been developed [155,156] more information is needed regarding the behavior of these connections under severe earthquake loading conditions. A comprehensive experimental research program is needed where these connections, as well as those already in use, will be studied under simulated seismic conditions. Meanwhile, it is recommended that designers have also been identified. The author considers these advances of paramount importance and would like to emphasize the need to present these advances to the profession. The author considers this to be more important than developing simple empirical rules for the design of standard elements. If the designer knows what the problems and their sources are, he has two possibilities for coping with them. First, he can try to avoid them. Since he knows the sources of the problems, if he cannot avoid them, he can try to minimize them by proper design, particularly proportioning and detailing. Two typical examples follow:

Most failures of R/C linear elements are caused by the development of high shear in flexural critical regions. The designer can avoid such problems by proper selection of structural form, selecting relatively slender members and/or using a low percentage of steel reinforcement of low yielding strength and strain hardening characteristics. Since such failures are due to sliding shear, designers can avoid or sufficiently delay the failure of such members by proper use and detailing of special web reinforcement in the flexural critical regions.

Another problem that has been observed in seismic behavior is the degradation in stiffness and strength of beam-column subassemblages with repeated cycles of deformation reversals. This problem occurs at the beam-column joints; its sources have been identified as high shear and/or high bond stress through the joint. The designer can avoid this problem by selecting wider columns, and beams with a low percentage of steel with low yielding strength and strain hardening characteristics. Or he can avoid the formation of beam plastic regions at the faces of the columns. If this cannot be done, proper detailing of the reinforcement of the beam, column, and joint can minimize the detrimental consequences of stiffness and strength degradation.

Following, with the presentation of the conclusions, there is a summary of advances in the design and understanding of seismic behavior of normal weight R/C elements and their cast-in-place subassemblages under 1D loading conditions. There has been very little research for 2D or 3D loading. However, some parameters influencing the seismic behavior of frame subassemblages under two dimensional-lateral motions have been identified.

There have been some significant advances in understanding behavior of lightweight concrete. Some of the peculiarities of this type of concrete have been identified by comparing its behavior with the behavior of similar normal weight concrete. These peculiarities include: a lower gain in strength and ductility with confinement (particularly with high strength [e.g. greater than 4,000 psi]); lower bond; and lower shear transfer. More comprehensive studies are needed of the mechanical characteristics of this type of concrete and its interaction with reinforcing steel under seismic conditions.

The amount of research in the area of prestressed and precast linear concrete elements has been small. However, there have been some advances in the proper use of prestressing, particularly for improving behavior of beam-column joints and columns in tall buildings. The main problem for precast construction is connection. Although many types of connections have been suggested, and some used, there is no available information about their behavior under seismic loading.

8.2 Conclusions

The following conclusions emphasize findings which have helped to advance the design and construction of normal weight R/C elements and their cast-in-place subassemblages subjected to 1D loading conditions. General observations

8. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS FOR FUTURE RESEARCH AND DEVELOPMENTS

8.1 Summary

Significant advances have been made in the last ten years in understanding seismic behavior of structural concrete linear elements and their connections. This improved understanding has had some impact in earthquake resistant design of R/C structures where these elements are used. However, much of present knowledge has not yet been practically applied. There are several problems in predicting seismic behavior of these linear elements and their connections. Some of these problems are of a general nature and apply to all types of elements, regardless of the material used (e.g. problems in predicting demand due to uncertainties about the ground motion and the overall response of the structure). There are other problems, inherent to the type of member and associated with the peculiar sensitivity of reinforced concrete construction to all those aspects which affect structural behavior - design, construction, maintenance, modification, and repair - which should be considered in order to obtain efficient seismic resistant construction.

Problems of a general nature have been discussed in section two. The seismic behavior of any element of a structure depends upon the interaction of the ground motion and the structure; there are many uncertainties in predicting both ground motion and structural response. All these uncertainties must be considered in order to judge the reliability of experimental results and to assess the implications of these results for design and construction of seismic resistant structures. To characterize these uncertainties properly, data from field and laboratory studies must be collected and statistically reviewed. Then studies may be carried out on the probability of failure of R/C elements.

Section two emphasizes the importance of loading history in the behavior of elements. The importance of properly selecting a structural layout and choosing the material to be used is also discussed.

The requirements for suitable seismic resistant structural materials are discussed. The relatively low value of strength per unit weight of normal weight concrete suggests the desirability of using lightweight concrete. The use of precast, partially prestressed lightweight aggregate concrete elements has tremendous potential in seismic resistant construction. However, the technology of lightweight aggregate; the problems of determining the optimum degree of prestressing; and the problems of connections of prefabricated elements, have not yet been resolved. Thus the most suitable R/C material for earthquake resistant design is still cast-in-place, ordinarily reinforced, normal weight concrete.

Section two also discusses the importance of studying the seismic behavior of basic structural components and their subassemblages, rather than the response of a whole structure.

A review of the inherent problems of linear reinforced concrete members and their connections shows that no general theory has been formulated to accurately predict the real seismic behavior (stiffness, strength, deformation, and energy dissipation capacities and their variation with load) of such structural components. It is doubtful that such a theory will ever be developed. However, there have been significant accomplishments in the understanding of such behavior, particularly of R/C elements that are used in plane moment-resisting frames subjected to unidirectional (1D) loading conditions in the plane. For these elements not only have the problems been determined, but the different sources of the problems

8.2.2 Columns. These elements are still the most susceptible to failure in destructive earthquakes, particularly when subjected to high axial and shear forces. This is because of the sensitivity of shear stress to variations in the values of many of the factors affecting such column stress.

(1) Short columns designed and constructed according to present U.S. seismic codes can dissipate moderate amounts of energy through inelastic deformations. This can be adequate for ductile moment-resisting frames which are properly designed, constructed, and maintained and in which the short columns are not subjected to significant fluctuations of axial force.

(2) In the case of large flexural ductility demands, the contribution of concrete to shear resistance should be ignored.

(3) Circular spiral is the most effective transverse reinforcement to confine concrete and prevent the main reinforcing bars from buckling.

(4) New types of column reinforcement have been developed in Japan. A combination of spiral and square hoops resulted in excellent hysteretic behavior.

(5) Because joint core behavior can lead to some damage of the concrete cover of the column, the column strength computation should be based on the strength of the core area only.

8.2.3 Beam-Column Joints. Design criteria have been forumlated for this type of joint. The criteria for the strength of the joint is that the beam-column joint should be the strongest and stiffest component of a basic moment-resisting frame subassemblage. While this usually has been so in the past, it might not be so in future structures, because while more stringent requirements for seismic design of beams and columns have recently been included in codes, no changes have been introduced for the design of joints. Research results have indicated that:

(1) The effectiveness of concrete to resist shear should only be considered when there is a compressive load on the column which exceeds $0.1f'_c A_g$.

(2) Vertical shear reinforcement should be provided to help transfer vertical shear force to complete the truss mechanism at the joint core. Vertical column bars should be used around the perimeter of the column section with spacing not exceeding six in. (150 mm).

(3) For exterior beam-column joints, if plastic hinging occurs in the beam at the column face it is recommedned that the diameter of the lontitudinal column bars should not exceed 1/25th or 1/20th of the beam depth (for 55 and 40 grade steel, respectively).

(4) For interior beam column joints, if plastic hinging occurs in the beams at the column face it is recommended that the maximum diameter of the longitudinal beam reinforcing bars should not exceed 1/35th or 1/25th of the column depth (for 55 and 40 grade steel, respectively). The diameter of longitudinal column bars are limited as for exterior joints.

(5) If plastic hinging occurs in the beam at the column face, in determining the anchorage length of beam steel it is necessary to distinguish between the effectiveness of the bond offered by unconfined concrete in the column cover (which is small and should be neglected) and that offered by the confined concrete core. In exterior joints, the anchorage should be considered to begin within the joint core at a distance of either one-half the column

applying to all members are presented first. Then observations for beams, columns and their connections are presented separately.

Reliable methods are lacking to predict demands, particularly deformation and energy dissipation demands, that can be expected during a structure's response to extreme earthquake shaking. Therefore, it is highly desirable to design R/C elements and their subassemblages so that they will be capable of dissipating the largest possible amount of energy through stable hysteretic behavior. Special attention should be paid to proportioning and detailing. The following recommendations are designed to achieve such stable, tough behavior.

8.2.1 Beams. Most of the following observations apply to the design of the potential beam critical regions.

(1) It is essential to provide sufficient shear capacity in potential critical (plastic hinge) regions to develop the required flexural deformation and energy dissipation capacities.

(2) Lower tension steel contents, ρ, are recommended than those presently allowed by R/C codes.

(3) It is recommended that beams be designed so that, at their connection with columns, they have a larger positive moment capacity than presently required by seismic codes ($\rho'/\rho \geq 0.75$ has been recommended).

(4) The location of splicing of main reinforcing bars should be carefully established. As much as economically feasible, curtailing of the main bars should be avoided.

(5) The effectiveness of different arrangements of transverse steel in confining concrete has been studied and constitutive laws for such confined concrete have been formulated.

(6) Present seismic code requirements for beam confinement are not adequate when large ductility is demanded.

(7) To prevent premature buckling of main reinforcing bars, each of these bars should be supported laterally by a corner of a tie and tie spacing should not exceed six bar diameters.

(8) The use of beams where the nominal unit shear stress, v_{max}, can exceed $6\sqrt{f'_c}$ (psi) ($0.5\sqrt{f'_c}$ (MPa)) should be avoided.

(9) Present code requirements result in satisfactory hysteretic behavior when v_{max} is $\leq 3\sqrt{f'_c}$ (psi) ($0.25\sqrt{f'_c}$ (MPa)).

(10) When v_{max} is in the range of $3\sqrt{f'_c}$ (psi) to $6\sqrt{f'_c}$ (psi) (($0.25\sqrt{f'_c}$ (MPa) to $0.5\sqrt{f'_c}$ (MPa)), it is necessary to use special web reinforcement. Although the use of intermediate longitudinal bars improves hysteretic behavior, the addition of diagonal reinforcement seems to be more effective in controlling sliding shear at critical regions.

(11) Conventional seismic resistant design is inadequate for coupling beams, of coupled shear wall systems, which have V_ud/M_u ratios of one or less. The energy dissipation capacity (ductility and useful stable strength) can be improved by placing the main reinforcement diagonally in the beams.

(1) Prestressed beams show marked elastic recoveries even after considerable inelastic deformations, leading to pinching of the hysteretic loops.

(2) Energy dissipation of prestressed concrete elements can be increased, and degradation of stiffness decreased, by the proper addition of bonded compression and transverse (confinement) reinforcements.

(3) Although high seismic loading rates of prestressed elements can result in strength increases of four to seven percent, and ductility increases of 10 to 15 percent, it is recommended that design computations can be based on static loading strengths only.

(4) The use of a reasonable level of prestressing through a central tendon improves hysteretic behavior of joint.

(5) The use of prestressing can improve behavior of ordinarily reinforced concrete exterior columns in tall slender buildings by decreasing the possibility of cracking due to tensile forces originated by overturning moments.

(6) Use of prestressing can improve the behavior of connections between linear elements.

(7) The use of prestressed and precast lightweight concrete structural elements has great potential for seismic resistant construction.

8.3 Recommendations for Future Research and Developments.

Among the different recommendations formulated in this report the following deserve special mention:

(1) Perform integrated analytical and experimental research on the three dimensional behavior of actual structures under realistic seismic loading conditions to determine the demands on different structural components. In order to carry out more realistic experiments than has been done up to date it is important to determine the expected loading or deformation histories that the structural elements will undergo. Seismic performance of R/C structures is very sensitive, not only to how the structures have been designed and detailed, but also to how they are constructed, and to the modifications, maintenance, and repair which they can undergo before an earthquake strikes. All these aspects must be considered in establishing design criteria.

(2) Improve quality control of the R/C materials. Statistical data regarding mechanical characteristics of the material from existing structures should be collected and studied.

(3) Perform experiments to improve prediction of the interface shear transfer in plastic hinge regions of beams and columns subjected to generalized loadings.

(4) Perform experiments under seismic loading conditions, on the contribution of the floor slab to: development of beam flexural capacity; behavior of the beam-column joint; and overall strength, stiffness, deformation, and energy dissipation capacity of basic frame subassemblages.

(5) Perform experiments to study the behavior of columns and beam-column joints subjected to two and three-dimensional loadings. Emphasis should be

depth or ten bar diameters, whichever is closer to the column face where the steel enters.

(6) Performance of exterior joints can be improved by using a beam stub at the far column face where the longitudinal beam bars can be anchored.

(7) Significant bond deterioration occurs at the joint core from load reversals cyclically applied to the beam bars. This results in beam fixed-end rotations, particularly when the stress applied to the beam bars entering the columns equals or exceeds yield.

(8) To avoid detrimental beam fixed-end rotations, beam hinges adjacent to column faces should be eliminated. Practical techniques to accomplish this have been suggested, tested, and proven to be satisfactory.

8.2.4 2D and 3D Loadings. The following observations are of a tentative nature, because of insufficient data.

(1) 2D column displacement ductility demands about twice as large as 1D ductilities are typical at a 1D displacement ductility of about five or more.

(2) To avoid difficulties under 2D it is recommended that frames be designed so that column displacement ductility demands under 1D are restricted to two.

(3) While compressive axial loads have little influence on column behavior under 2D loading, tensile axial loads substantially reduce the stiffness and shear capacity at low loads.

(4) Theoretically, for a symmetrical two-way frame, joint design for biaxial shear leads to approximately twice the shear required for uniaxial shear design. Because this can create serious practical problems, it is suggested that beam hinges adjacent to column face be eliminated.

8.2.5 Use of Lightweight Aggregate Concrete. Because of the relatively meager data available, the following observations are of a preliminary nature.

(1) The effectiveness of the confinement, bond and shear transfer of lightweight aggregate concrete is inferior to that of normal weight aggregate concrete of similar strength. The higher the strength of the concrete the larger the difference in confinement effectiveness. Furthermore, lightweight aggregate concrete has higher creep. Therefore caution should be used in applying equations or seismic code provisions derived for normal weight aggregate to lightweight aggregate concrete, particularly in designing columns.

(2) Under cyclic loading, the energy dissipated by beam-column subassemblages cast of lightweight aggregate concrete is significantly smaller than that of similar normal weight concrete subassemblages.

(3) The compressive strength of lightweight aggregate concrete used in seismic resistance construction should be limited according to the mechanical characteristics of the aggregate.

8.2.6 Use of Prestressed and Precast Techniques. In addition to the problems common to any kind of reinforced concrete elements, the main findings of the reviewed research are:

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placed on the effects of high shears and the fluctuation from high compressive to high tension axial forces.

(6) Conduct statistical studies of the variation of v^R_{max} / v^C_u in elements of existing buildings.

(7) Perform experimental studies of behavior of lapped and mechanical splices under high intensity load reversals and at different loading (strain) rates.

(8) Perform experiments to study behavior of construction joints in beams and columns.

(9) Perform experimental studies to establish reliable bond-slippage constitutive law for the beam's reinforcing bars along the confined concrete of beam-column joints.

(10) Conduct analytical studies of how the fixed-end rotations at the beam ends of column faces affects seismic response of framed structures.

(11) Conduct integrated experimental and analytical studies on the seismic behavior of reinforced lightweight aggregate concrete elements, with emphasis on: the effectiveness of confinement, bond, and shear transfer of such concrete; the higher rate of creep for lightweight than for similar normal weight; and how that higher creep can effect the seismic performance of framed structures.

(12) Conduct coordinated analytical and experimental studies to define: the degree of stiffness; damping; abruptness of failure; and hysteretic behavior of prestressed concrete subassemblages. These subassemblages should contain combinations of prestressed tendons and deformed bar reinforcements similar to those likely to be found in practice.

(13) Make generic studies of hysteretic behavior of different types of connections between precast elements. These studies should cover non-tensioned and post-tensioned connections subjected to loading intensities and histories similar to those which would exist during extreme earthquakes. These studies should examine precast elements of various types (particularly lightweight prestressed) and various cross sections.

(14) Conduct research programs which examine the applicability of reduced ductility and strength requirements for areas other than those of highest seismicity.

It is hoped that this report can serve as a basis for spirited discussions at the Symposium, and that these discussions will contribute toward the solution of the many problems and questions that have been raised here. Because of the complex nature of these problems, international collaboration is needed between practitioners, educators, researchers, and representatives from industry and government agencies in the field of earthquake resistant construction.

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AVANCES EN EL DISEÑO SISMICO DE ESTRUCTURAS DE CONCRETO REFORZADO

Procedimiento de Diseño por Resistencia Ultima en Estructuras de Concreto Reforzado

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INTRODUCCION.

Como principio básico del diseño antisísmico de estructuras arquitectónicas en Japón, se plantea la formación de un mecanismo de falla o mecanismo de fluencia, como el formado por aparición de articulaciones plásticas en vigas, o el mecanismo de columna fuerte - viga débil. La aparición de articulaciones plásticas en columnas se contempla únicamente en la parte inferior de las columnas del primer nivel y en la parte superior de las del último nivel. El mecanismo de articulaciones plásticas en vigas se plantea con el objeto de incrementar la capacidad de disipación de energía en la estructura, así como lograr una distribución uniforme de dicha disipación.

El criterio básico de diseño antisísmico se resume en la Tabla-1 Este consta de dos fases, que esencialmente corresponden a los dos niveles mostrados en la tabla. La primera fase de diseño tiene por objeto proteger las "partes débiles" de la estructura, esto es, procura eliminar la formación de articulaciones plásticas (propias del mecanismo propuesto) ante un sismo correspondiente al nivel 1. La segunda parte del procedimiento de diseño, tiene por objeto asegurar la formación del mecanismo de fluencia planteado ante un sismo correspondiente al nivel 2. La carga de falla o fluencia, asociada con la formación del mecanismo, se calcula en forma similar a la definición de capacidad por carga última estructural definida en el reglamento de construcciones arquitectónicas Japonés. A continuación se presenta un resumen a grandes rasgos del criterio de diseño correspondiente al nivel 2 de la Tabla-1.

Tabla-1 Criterios de diseño antisísmico [1]

Nivel de riesgo sísmico	Nivel 1	Nivel 2
Probabilidad de ocurrencia	uno en la vida útil	máximo posible
Máximas velocidades de terreno	25 cm/s	50 cm/s
Fuerzas en los elementos	Agrietamiento en el concreto sin fluencia en el acero	Fluencia en el acero sin falla total de la estructura
Ductilidad por piso	menor que 1	menor que 2
Ductilidad en elementos	menor que 1	menor que 2
Angulo de deformación de piso	menor que 1/200	menor que 1/100

1. ESTRUCTURACION.

1.1 Conceptos base para la estructuración.

Definición del mecanismo de fluencia, con el propósito de que se genere el mismo al alcanzar la estructura su resistencia de diseño (la carga de falla o fluencia). Definición de las características de los elementos estructurales, con objeto de verificar que los elementos en los que se proyecta la aparición de articulaciones plásticas tengan la resistencia y capacidad de deformación adecuadas. Asimismo, que

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aquellos elementos en los que no se proyecta la aparición de articulaciones plásticas, tengan la resistencia adecuada Definición de las características en plano y elevación de la estructura. Para evitar posibles comportamientos no deseados en la estructura, se verifica la uniformidad en la distribución de rigideces, resistencias y ductilidades entre otros aspectos.

1.2 Estructuras a base de marcos momento-resistentes.

El mecanismo de fluencia y la ubicación de las articulaciones plásticas se proyecta para que las articulaciones plásticas se formen en los extremos de las vigas de todos los niveles y en la parte inferior de las columnas del primer nivel, formando un "mecanismo de fluencia por vigas" (Fig.1).

Excepción en los mecanismos de fluencia y en la ubicación de las articulaciones plásticas. Respecto al mecanismo planteado, se permiten la aparición de articulaciones plásticas como se indica:

a) La parte superior de las columnas del último nivel.
b) Columnas exteriores cuya carga axial decremente ante la incidencia de fuerzas sismicas.
c) Columnas interiores que no intervengan en la transferencia de fuerza sismica incidente.

1.3 Estructuras con muros estructurales.

Básicamente en una estructura con muros estructurales se busca la simetría en su posición, la regularidad y uniformidad en el plano del mismo, y la continuidad del muro desde la cimentación en toda la altura de la estructura.

En caso de emplear muros estructurales con aperturas en el plano, el efecto de estas en la rigidez y resistencia del mismo deberá ser considerado.

Respecto a la ubicación de articulaciones plásticas por flexión en muros estructurales, se proyecta su formación en la parte inferior del muro en el primer nivel (Fig.2). Sin embargo, también se permite el giro del muro y la aparición de articulación plástica en la trabe de cimentación (Fig.3).

La parte de la estructura a base de marcos momento-resistentes, sigue el mismo criterio presentado en la sección 1.2. Sin embargo, si se garantiza que el muro estructural cuenta con una resistencia adecuada, se permitirá la formación de articulaciones plásticas en las columnas.

1.4 Estructura de cimentación.

Como regla general en la trabe de cimentación no se proyectará la formación de articulación plástica. Sin embargo, cuando se define un mecanismo de fluencia que presente giro en la base de un muro estructural (Fig.3), la trabe de cimentación requerirá ser diseñada para presentar articulación plástica en la vecindad al muro

Respecto a la losa de cimentación y a los pilotes o pilas, no se permitirá la formación de articulaciones plásticas en los mismos. Las estructuras de sótanos deberán revisarse a ser suficientemente rígidas, y no se permitirá la formación de articulaciones plásticas en ningún elemento estructural de los mismos.

1.5 Elementos no estructurales.

Las juntas entre la estructura y los elementos no estructurales deberá hacerse de tal manera que el comportamiento de estos no afecte a la formación del mecanismo de fluencia definido. Deberán diseñarse para evitar falla o caída de los mismos ante sismos de mediana intensidad.

2. METODO DE DISEÑO.

2.1 Procedimiento de diseño.

El diseño estructural tendrá por objeto asegurar que ante carga vertical y sismos de mediana intensidad la estructura tenga y mantenga la resistencia y funcionalidad adecuada, y ante sismos de gran intensidad asegurar que la estructura tenga la ductilidad o capacidad de deformación necesaria para desarrollar el mecanismo de fluencia ante la fuerza lateral incidente sin presentar la falla total.

El diseño ante carga vertical contempla la revisión de resistencia, deformación, desplomes, agrietamientos y posibilidad de problemas de vibración no deseada. El diseño ante carga lateral se lleva a cabo en dos partes, primero el diseño del mecanismo de fluencia, y segundo el diseño para el aseguramiento de la formación del mecanismo de fluencia. Estos se resumen como sigue:

a) En el diseño del mecanismo de fluencia, se verificará que la estructura tenga la resistencia ante carga lateral adecuada, y que los elementos presenten ductilidades dentro de los límites requeridos. Para lo cual se hacen análisis elasto-plástico de la estructura considerando la resistencia esperada de los elementos estructurales.

b) Para asegurar la formación del mecanismo de fluencia ante un sismo de gran intensidad, se realiza un análisis elasto-plástico que muestre que, en elementos en los que no se proyectó la formación de articulaciones plásticas, no se presente falla o formación de articulaciones plásticas. Para llevar a cabo este tipo de análisis, se considera la resistencia esperada para los elementos en los que no se proyectó la formación de articulaciones plásticas, y se considera el límite superior de resistencia en aquellos elementos en los que se proyectó la formación de articulaciones plásticas para la conformación del mecanismo de fluencia. El significado de cada término se ve en la Fig 4.

2.2 Combinación de estados de carga.

Las cargas a considerar en diseño son cargas muertas, vivas, por acumulación de nieve, por viento y las debidas a sismo. Las condiciones de carga a considerar, así como los factores que afectan a cada una de ellas varían de acuerdo a la región y país Sin embargo, el mayor número de combinaciones posibles debe considerarse en el diseño.

2.3 Diseño del mecanismo de fluencia.

Fuerzas sísmicas de diseño:

(a) El coeficiente de fuerza cortante basal de diseño se calcula de la siguiente manera

$$C_1 = Z\, R_t\, C_B \qquad (1)$$

donde, C_1: coeficiente de fuerza cortante basal; Z. coeficiente sísmico zonal; R_t: factor por características dinámicas de la estructura; C_B coeficiente de cortante basal estándar. El coeficiente de cortante basal estándar se considera igual o mayor a 0.25 para estructuras a base de marcos momento-resistentes, y se considera igual o mayor a 0.30 para estructuras a base de muros estructurales.

(b) Para calcular las fuerzas sísmicas de diseño, se parte de la hipótesis de que el comportamiento de los ejes principales ortogonales de la estructura es independiente. La fuerza sísmica horizontal a aplicarse en cada nivel se puede calcular como se indica (salvo investigación específica)

$$\begin{aligned} F_i &= P_i + P_t \qquad (\text{para } i = n) \\ &= P_i \qquad\quad\;\; (\text{para } i \leq n - 1) \end{aligned} \qquad (2)$$

donde, P_t: fuerza lateral concentrada en el último nivel, que se calcula con la expresión (3) Para estructuras con menos de 6 niveles $P_t = 0$.

$$P_t = \alpha\, T\, Q_1 \qquad (3)$$

El valor de P_i se calcula como se indica

$$P_i = (Q_1 - P_t)\, W_i\, H_i \,/\, (\Sigma\, W_i\, H_i) \qquad (4)$$

donde, F_i : Fuerza horizontal concentrada en la losa del nivel (i + 1)
Q_1 : Fuerza cortante de entrepiso para diseño del primer nivel, $Q_1 = C_1\, \Sigma\, W_i$
α : Coeficiente de la carga concentrada en el último nivel, igual a 0.10
T : Período fundamental de la estructura (s), $T = 0.02\, H_n$
H_i . Altura respecto al nivel de suelo de la losa del nivel (i + 1)

H_n : Altura de la estructura (m)
W_i : Suma de carga muerta y carga viva (para sismo) en el nivel (i + 1)
n : Número de niveles de la estructura (siendo 1 el nivel de suelo)

Análisis lineal.

El estado de esfuerzos a emplear para el diseño del mecanismo de fluencia se calcula considerando la rigidez de los elementos estructurales componentes y haciendo un análisis lineal de la estructura con las siguientes hipótesis:

(a) Elementos en los que se proyecta la formación de articulaciones plásticas, la rigidez se considerará igual a la rigidez secante al punto de fluencia. Elementos en los que no se proyecta la formación de articulaciones plásticas, al calcular su rigidez se considerarán los efectos de agrietamiento por flexión.

(b) En el caso de muros estructurales o elementos con relación claro/peralte pequeña, los efectos de deformación por cortante deberán ser considerados.

(c) Las losas de cada nivel se considerarán como elementos rígidos.

Redistribución de esfuerzos

Para definir el estado de esfuerzos a emplear en el diseño del mecanismo de fluencia, el estado de esfuerzos obtenido del análisis lineal puede ser redistribuido con las condiciones siguientes:

(a) Posterior a la redistribución de esfuerzos, deberán satisfacerse las condiciones de equilibrio.

(b) La variación del valor de los momentos por la redistribución respecto a los valores obtenidos por el análisis lineal, no será mayor que el 20% para estructuras a base de marcos y 25% para estructuras a base de muros.

(c) La variación por la redistribución de la suma de momentos de entrepiso, no deberá ser mayor a 5% de la suma de momentos de entrepiso producto del análisis lineal en la estructura a base de marcos, y no mayor que 15% en estructuras a base de muros.

Límites de deformación.

La deformación angular de entrepiso de una estructura ante un análisis sísmico lineal deberá limitarse a ser menor que 1/200 rad

2.4 Diseño para aseguramiento de la formación del mecanismo de fluencia.

Estado de esfuerzos para diseño.

El estado de esfuerzos para asegurar la formación del mecanismo de fluencia se obtendrá empleando el límite superior de resistencia en los elementos que se proyecta la formación de articulación plástica. Se lleva a cabo un análisis pseudo-estático no-lineal, incrementando la fuerza lateral hasta la formación del mecanismo de fluencia El estado de esfuerzos obtenido de este análisis se modificará considerando los siguientes efectos.

(a) Efecto del comportamiento dinámico

(b) Efecto de incidencia de fuerza sísmica en dos direcciones

Análisis no-lineal.

Tomando en cuenta las características de comportamiento elásto-plástico de los elementos estructurales, se realiza el análisis pseudo-estático no-lineal para determinar el estado de esfuerzos a la formación del mecanismo de fluencia. Adicionalmente, se considerarán las siguientes hipótesis:

(a) La distribución de la fuerza sísmica lateral será en forma de triángulo invertido. Por medio de métodos paso a paso o de trabajo virtual, se resolverá la ecuación de equilibrio.

(b) En el cálculo del límite superior de resistencia de los elementos donde se proyecta la formación de articulaciones plásticas, se empleará el acero de refuerzo propuesto en el prediseño.

(c) Las losas de cada nivel se considerarán como elementos rígidos.

(d) La rigidez no-lineal del elemento se calculará en base a la rigidez elástica y al refuerzo del mismo. Para materiales comúnmente empleados se puede emplear la ecuación propuesta por

Sugano-Aoyama[2]

$$K_y = \alpha_y K_o$$
$$\alpha_y = (0.043 + 1.64\, \eta\, p_t + 0.043\, a / D + 0.33\, \eta_o)(d / D)^2$$
$$\eta_o = N / b D \sigma_B \qquad (5)$$

donde, K_y: rigidez secante al punto de fluencia por flexión del elemento; K_o: rigidez elástica del elemento; η: E_s/E_c, E_s y E_c son los módulos de Young del acero y concreto respectivamente; p_t: Porcentaje de acero de refuerzo en la sección; d: peralte efectivo del elemento; D: peralte total de la sección, a/D: Relación entre claro de cortante y peralte, N: fuerza axial de compresión en el elemento; σ_B: Resistencia a la compresión del concreto

Amplificación de esfuerzos por efecto del comportamiento dinámico.

El coeficiente de incremento de momento y fuerza cortante por efecto del comportamiento dinámico en columnas y muros estructurales, a excepción de que se determine por medio de una investigación especial, se calcularán de la siguiente manera:

$$\omega_{ci} = 1.0 + (\Delta\omega_i / \phi_o)(\beta_{chi} / \beta_{ci}) \qquad (6)$$
$$\omega_{wi} = 1.0 + (\Delta\omega_i / \phi_o)(\beta_{whi} / \beta_{wi}) \qquad (7)$$

$\Delta\omega_i$ es el parámetro con que se considera el efecto de modos superiores en el nivel i, y se calcula

$$\begin{aligned} \Delta\omega_i &= 0.25 && (\text{para } i = 1) \\ &= 0.20 && (\text{para } 2 \leq i \leq n/2) \\ &= 0.20 + 0.10 (i - n/2) && (\text{para } i > n/2) \end{aligned} \qquad (8)$$

donde, ω_{ci}, ω_{wi}: factores de amplificación de esfuerzos en columnas y/o muros del nivel i; ϕ_o: factor de incremento de resistencia estructural por el límite superior de resistencia de los elementos ($=C_{1o}/0.25$); C_{1o}: Coeficiente de cortante basal al momento de formación del mecanismo de fluencia, al emplear el límite superior de resistencia en los elementos; β_{ci}, β_{wi}: porcentaje de fuerza cortante a resistir por efecto del modo fundamental en columnas y/o muros del nivel i; β_{chi}, β_{whi}. porcentaje de fuerza cortante a resistir por efecto de modos superiores en columnas y/o muros del nivel i.

Amplificación por efecto de incidencia sísmica biaxial.

En el caso de columnas, el momento y/o el cortante de diseño se afectará por el factor de amplificación por comportamiento dinámico ω_{ci}, y este a su vez se afectará por el factor de seguridad ante efecto sísmico biaxial ψ_2. El factor de seguridad ante efecto sísmico biaxial ψ_2 se tratará independientemente para cada eje de análisis, y por regla se considera igual a 0.10. La carga axial de diseño en columnas y muros se calculará considerando el 100% de los resultados del análisis plano en una dirección, adicionando el 50% de los resultados del análisis plano en la dirección ortogonal.

Deformación de seguridad estructural.

Los elementos estructurales en los que se proyecta la formación de articulaciones plásticas deberán diseñarse de modo que la capacidad de deformación plástica supere la deformación de seguridad estructural. La deformación de seguridad estructural de los elementos tiene relación directa con la deformación por seguridad estructural de la estructura en su conjunto, por lo tanto se obtendrá del análisis pseudo-estático no-lineal.

3. DISEÑO DE ELEMENTOS A FLEXION Y CARGA AXIAL.

3.1 Determinación de las secciones transversales.

Hipótesis básicas para la determinación de la resistencia última a flexión.

+ Las deformaciones en el concreto y el acero en cualquier punto de la sección transversal serán proporcionales a la distancia de dicho punto al eje neutro.
+ La relación esfuerzo-deformación del acero de refuerzo, tanto a tensión como a compresión, se considerará elástica para valores menores que los ilustrados en la Tabla-2. Para deformaciones mayores que las correspondientes a los esfuerzos ilustrados en la Tabla-2, el esfuerzo en el acero de refuerzo se tomará igual a la resistencia a la fluencia del material.
+ La relación esfuerzo-deformación del concreto se obtendrá empleando modelos adecuados (vrg. el modelo de Kent - Park [3], Fig.5)

3.2 Resistencia esperada a la flexión.

La resistencia esperada a la flexión se calculará en base a las hipótesis que se indican:

(a) La deformación del concreto en la fibra exterior a compresión de la sección transversal se considerará como 0.003. Para la resistencia del acero de refuerzo se considerarán los valores mostrados en la Tabla.2

Tabla.2 Resistencia del acero de refuerzo

Tipo de acero de refuerzo	Resistencia del material (a) Resistencia esperada	(b) Límite superior de resistencia
corrugado 3000	1.0 σ_y	1.30 σ_y
corrugado 3500	1.0 σ_y	1.25 σ_y
corrugado 4000	1.0 σ_y	1.25 σ_y
corrugado alta	1 0 σ_y	-------

σ_y: límite inferior de fluencia probado del acero de refuerzo

(b) El acero de refuerzo de losa que tenga la longitud de anclaje requerida y se encuentre en el ancho efectivo a considerar de la losa, podrá tomarse como parte del acero a tensión en su caso.

(c) Podrá considerarse el acero de refuerzo colocado en varias capas.

3.3 Límite superior de resistencia a flexión.

El límite superior de la resistencia a flexión se calculará en base a las siguientes hipótesis:

(a) La deformación del concreto en la fibra exterior a compresión de la sección transversal se tomará como 0 003 (un ejemplo en vigas se ilustra en la Fig.6). El límite superior de resistencia del acero de refuerzo se considerará conforme se indica en la Tabla.2

(b) Cuando se traten secciones T o I, el ancho efectivo correspondiente a los patines se tomará como dos veces el considerado para el cálculo de la resistencia esperada. Igualmente, el acero de refuerzo de losa o muro se considerará en el cálculo.

(c) El acero de refuerzo colocado en capas, o bien cualquier refuerzo que contribuya a resistir esfuerzos por flexión, deberán considerarse en el cálculo.

3.4 Límite de la carga axial permisible en elementos con posible formación de articulación plástica.

En elementos en los que por diseño se proyecta la formación de articulación plástica en el mecanismo de fluencia, deberán satisfacerse los siguientes límites requeridos para la carga axial.

(a) La carga axial en columnas deberá satisfacer la ecuación (9)

$$-k_2 A_g \sigma_y \leq N_c \leq k_1 A_c \sigma_B \qquad (9)$$

donde, N_c: carga axial de compresión en la columna, obtenida del diseño para aseguramiento de la formación del mecanismo de fluencia; A_c: Area de la sección transversal de la columna; A_g: area total del acero de refuerzo longitudinal a tensión efectivo en la columna; σ_y: resistencia esperada del acero de refuerzo longitudinal; k_1 coeficiente por carga axial de compresión (=1/3).

En caso de cumplir los requisitos para confinamiento del acero lateral, k_1 puede considerarse como 2 / 3); k_2: coeficiente por carga axial de tensión (= 3/4).
(b) La carga axial en muros, por lo general deberá satisfacer la ecuación (10)

$$N_\omega \leq k_3 \, A_{core} \, \sigma_B - A_{\omega s} \, \sigma_{\omega yu} \qquad (10)$$

donde, N_ω. carga axial de compresión en el muro obtenida del diseño para aseguramiento de la formación del mecanismo de fluencia; A_{core}: area de la sección transversal de la columna ubicada en el extremo a compresión del muro; $A_{\omega s}$, $\sigma_{\omega yu}$: area y límite superior de resistencia del acero vertical colocado en la parte del muro; k_3 = 2/3 (en caso de cumplir los requisitos de confinamiento para el acero lateral, el valor de k_3 se puede considerar igual a la unidad).

3.5 Regulación sobre la estructuración.

Vigas.

El ancho de la viga será mayor a 25 cm. Para vigas en las que se proyectó la formación de articulaciones plásticas, el ancho de las mismas será mayor a 1/4 veces su peralte.

El acero de refuerzo longitudinal será acero corrugado con diámetro mayor a 19 mm.

En vigas con articulaciones plásticas, el porcentaje de acero de tensión p_t, incluyendo la contribución del acero de la losa, será menor a 0.025. El area total del acero de refuerzo a compresión será mayor a 0.5 veces el area total del acero de refuerzo a tensión. ($p_t = A_t / b \, d$, A_t: area total del acero de refuerzo a tensión, b: ancho de la viga, d: peralte efectivo de la viga).

La colocación del acero de refuerzo podrá hacerse en dos capas máximo.

Columnas.

Cualquier de los lados o diámetro de columna deberá ser mayor a 40 cm. Para columna con formación de articulación plástica proyectada, el lado corto de columna será mayor a 1/3 veces el lado largo de la misma.

El acero de refuerzo longitudinal será acero corrugado con diámetro mayor a 19 mm.

Muros.

La geometría transversal del muro estructural será en forma de I con columnas en los extremos. El ancho del muro será el valor mayor entre 15 cm y 1 / 20 de la altura de entrepiso.

El acero de refuerzo a emplear en el muro será corrugado con diámetro mayor a 10 mm, se requiere que las cantidades de acero horizontal y vertical sean iguales. En zona de articulación plástica el acero de refuerzo deberá colocarse doble (en dos capas).

En el caso de proyectar una abertura en zona de articulación plástica, la abertura deberá hacerse al centro del muro lo más posible. La dimensión del hueco será tal que no provoque pérdida de monoliticidad en el muro.

4. DISEÑO ANTE FUERZA CORTANTE.

4.1 Método de diseño.

4.1.1 Fundamentos básicos del procedimiento de diseño.

El procedimiento de diseño ante fuerza cortante tiene por finalidad que la resistencia esperada de los elementos ante cortante, sea mayor que la fuerza cortante considerada en el diseño para aseguramiento de la formación del mecanismo de fluencia. También, para que los elementos donde se proyecta la formación de articulaciones plásticas tengan una capacidad de deformación plástica que supere al límite de deformación estructural para el mecanismo de fluencia. En el caso de columnas y vigas, se corrobora que la resistencia por adherencia del acero de refuerzo longitudinal sea mayor que el estado de esfuerzos de adherencia en el mismo, obtenido en el diseño para aseguramiento de la formación del mecanismo de fluencia.

4.1.2 Resistencia del acero de refuerzo por cortante.

En el cálculo de la resistencia por cortante, la resistencia del acero de refuerzo lateral deberá considerarse igual a la resistencia esperada del mismo.

4.2 Resistencia ante cortante de columnas y vigas.

La expresión para calcular la resistencia esperada ante cortante en columnas y vigas es como sigue

$$V_u = b\, j_t\, p_w\, \sigma_{wy} \cot\phi + \tan\theta\, (1 - \beta)\, b\, D\, \nu\, \sigma_B / 2 \qquad (11)$$

donde,

$$p_w\, \sigma_{wy} \leq \nu\, \sigma_B / 2$$

$$\tan\theta = [(L / D)^2 + 1]^{1/2} - L / D$$

$$\beta = (1 + \cot^2\phi)\, p_w\, \sigma_{wy} / (\nu\, \sigma_B)$$

$$\sigma_{wy} \leq 25\, \sigma_B$$

σ_{wy}: resistencia del acero de refuerzo por cortante; b, jt, D, L: son el ancho del elemento, distancia entre el centroide de los aceros de refuerzo a tensión y compresión, peralte total, y claro libre del elemento, respectivamente; pw: porcentaje del acero de refuerzo por cortante. ν es el coeficiente de resistencia a la compresión efectiva del concreto. ϕ representa el ángulo de inclinación de la zona de concreto a compresión componente del mecanismo de armadura (Fig.4.a). θ representa la inclinación del mecanismo de arco (Fig.4.b). Los valores de ν y ϕ se determina como se indica:

(a) Elementos estructurales en los que no se proyecta la formación de articulación plástica

$$\nu = 0.7 - \sigma_B / 2000 \qquad (12)$$

$$\cot\phi = \min\{\, 2.0,\ j_t / (D \tan\theta),\ [\nu\, \sigma_B / (p_w\, \sigma_{wy}) - 1]^{1/2} \,\} \qquad (13)$$

(b) Para elementos en los que se proyecta la formación de articulación plástica al formarse el mecanismo de fluencia, el coeficiente de resistencia a la compresión efectiva del concreto ν, se calculará conforme a la ecuación (14). El valor de cotϕ se obtendrá como el menor valor de los calculados empleando las ecuaciones (13) y (15). Sin embargo, en estas expresiones el valor de β será calculado empleando el valor de cotϕ correspondiente a la zona exterior a la articulación plástica, y el valor de pw σ_{wy} correspondiente a la zona de articulación plástica.

$$\begin{aligned} \nu &= (1.0 - 15\, R_p)(0.7 - \sigma_B / 2000) && 0 < R_p \leq 0.05 \\ &= 0.25\,(0.7 - \sigma_B / 2000) && 0.05 < R_p \qquad (14) \end{aligned}$$

$$\begin{aligned} \cot\phi &= 2\ 0 - 50\, R_p && 0 < R_p \leq 0.02 \\ &= 1.0 && 0.02 < R_p \qquad (15) \end{aligned}$$

donde, Rp deformación angular de la articulación plástica del elemento correspondiente a la deformación por seguridad estructural del elemento en todo su conjunto.

En la zona que tendrá comportamiento en el rango elástico, para elementos donde se proyecta la formación de articulación plástica, el cálculo de la resistencia al cortante se hará empleando el coeficiente de resistencia a la compresión efectivo del concreto calculado conforme la expresión (14). El valor de cotϕ se tomará como el menor de los calculados conforma las ecuaciones (13). Sin embargo, el valor de β se tomará igual al empleado para la zona de articulación plástica

La confiabilidad del procedimiento de diseño presentado, se aprecia en la Fig.8, donde se comparan resistencias calculadas con resultados experimentales.

4.3 Resistencia ante cortante en muros estructurales.

La resistencia esperada a cortante de un muro estructural se puede calcular con la expresión (16)

$$V_u = t_w \, l_{wb} \, p_s \, \sigma_{sy} \cot\phi + \tan\theta \, (1 - \beta) \, t_w \, l_{wa} \, \nu \, \sigma_B / 2 \qquad (16)$$

donde,

$$p_s \, \sigma_{sy} \leq \nu \, \sigma_B / 2$$

$$\tan\theta = \{ [(h_w / l_{wa})^2 + 1]^{1/2} - h_w / l_{wa} \}$$

$$\beta = (1 + \cot^2\phi) \, p_s \, \sigma_{sy} / (\nu \, \sigma_B)$$

σ_{sy}: resistencia del acero de refuerzo por cortante del muro ($\sigma_{sy} \leq 4000$ kgf/cm^2); t_w: ancho del muro; p_s: porcentaje de refuerzo por cortante del muro, h_w: altura de diseño del muro (puede considerarse igual a la altura de entrepiso); ϕ: ángulo de inclinación de la zona de concreto a compresión en el mecanismo de armadura del muro ($\cot\phi = 1.0$); l_{wb}, l_{wa}: longitudes equivalentes del muro a considerar en los mecanismos de armadura y arco respectivamente.

En el cálculo de las longitudes equivalentes de muro para ambos mecanismos, la contribución de las columnas laterales puede considerarse, y se calculan como se indica:

$$l_{wa} = l_w' + D_c + \Delta l_{wa} \qquad (17)$$

$$l_{wb} = l_w' + D_c + \Delta l_{wb} \qquad (18)$$

donde, l_w': longitud del muro comprendido entre las columnas laterales; D_c. Peralte de las columnas laterales; Δl_{wa}, Δl_{wb}: incremento de la longitud efectiva de muro que se valúa conforme las expresiones (19) y (20)

$$\begin{array}{lll} \Delta l_{wa} = A_{ce} / t_w & A_{ce} \leq t_w D_c & \\ \quad\;\;\; = [D_c + (A_{ce} D_c / t_w)^{1/2}] / 2 & A_{ce} \succ t_w D_c & (19) \\ \Delta l_{wb} = A_{ce} / t_w & A_{ce} \leq t_w D_c & \\ \quad\;\;\; = D_c & A_{ce} \succ t_w D_c & (20) \end{array}$$

A_{ce}: Area de la sección transversal de la columna lateral, a calcular según la expresión (21).

$$A_{ce} = A_c - N_{cc} / \sigma_B \qquad A_{ce} \leq 3 \, t_w D_c \qquad (21)$$

A_c: area de la sección transversal de la columna lateral sujeta a compresión; N_{cc}: carga axial en la columna lateral del nivel superior a diseñar, obtenida del análisis de esfuerzos para el diseño por aseguramiento de formación del mecanismo de fluencia.

Para cuantificar el coeficiente de resistencia a la compresión efectiva del concreto para diseño ante fuerza cortante en muros estructurales, se plantea el siguiente procedimiento:

En la zona donde no se prevé comportamiento plástico del concreto en el muro, ν se valuará conforme la expresión (12). Para la región o el elemento donde se proyecta la formación de articulación plástica, el valor de ν para el cálculo de resistencia al cortante se tomará como sigue:

$$\begin{array}{lll} \nu = 0.7 - \sigma_B / 2000 & R_u \prec 0.005 & \\ \;\;\; = (1.2 - 40 \, R_u) (0.7 - \sigma_B / 2000) & 0.005 \leq R_u \prec 0.02 & \\ \;\;\; = 0.4 (0\ 7 - \sigma_B / 2000) & 0.02 \leq R_u & (22) \end{array}$$

donde, R_u. deformación de seguridad estructural del muro

En muros estructurales, el porcentaje mínimo de refuerzo ante fuerza cortante no será menor a 0 0025. Para las columnas laterales, en elementos donde se proyecta la formación de articulación plástica, el porcentaje de acero de refuerzo ante cortante no será menor a 0.003.

5. DISEÑO POR ADHERENCIA ENTRE ACERO DE REFUERZO Y CONCRETO.

5.1 Esfuerzos de adherencia empleados para diseño.

Los esfuerzos de diseño por adherencia entre el acero de refuerzo y el concreto circundante, se calculan empleando la expresión (23). Sin embargo, para elementos donde se proyecta la formación de articulación plástica en un solo extremo, o no se proyecta la formación de ninguna, los esfuerzos por adherencia serán el valor menor proporcionado por las expresiones (23) y (24).

$$\tau_f = d_b \ \Delta\sigma / [\ 4\ (\ L\ -\ d\ )\ ] \qquad (23)$$

$$\tau_f = b\ p_{wt}\ \sigma_{wy} \cot\phi\ /\ \Sigma\psi \qquad (24)$$

donde, $\Delta\sigma$ es la diferencia del estado de esfuerzos en ambos extremos del acero de refuerzo longitudinal obtenido durante el diseño para aseguramiento de formación del mecanismo de fluencia. Cuando en ambos extremos se presente articulación plástica $\Delta\sigma = 2\ \sigma_{yu}$, cuando solo en un extremo se presente articulación plástica $\Delta\sigma = \sigma_{yu} + \sigma_y$, finalmente, en el caso de no presentarse articulación plástica $\Delta\sigma = 2\ \sigma_y$. Aquí, σ_{yu} y σ_y son el límite superior de resistencia y la resistencia esperada, respectivamente, del acero de refuerzo longitudinal. También, d_b : diámetro del acero de refuerzo; $\Sigma\psi$: sumatoria del perímetro de todo el acero de refuerzo. L, b, d : longitud o claro libre del elemento, ancho y peralte efectivo del elemento, respectivamente; p_{wt}, σ_{wy}, ϕ : porcentaje de acero de refuerzo empleado para el diseño ante cortante en el centro del claro del elemento, resistencia del acero de refuerzo de cortante, ángulo de inclinación del concreto a compresión componente del mecanismo de armadura en el elemento, respectivamente.

5.2 Resistencia de adherencia.

La resistencia ante problemas de adherencia en columnas y/o vigas estructurales se puede cuantificar empleando la expresión (30). Sin embargo, para el refuerzo longitudinal colocado en la parte superior de vigas, el valor calculado con la expresión (30) será penalizado 0.80 veces.

$$\tau_{bu} = (\ 1\ 2\ +\ 5\ p_w'\ b\ /\ d_b\ )\ (\ \sigma_B\ )^{1/2} \qquad (30)$$

donde, p_w' porcentaje de acero de refuerzo lateral colocado en la perímetro exterior de la sección transversal.

6. UNION VIGA - COLUMNA.

6.1 Objetivo del procedimiento de diseño.

La unión viga - columna se diseñará con el propósito de no presentar falla durante la formación del mecanismo de fluencia hasta alcanzar la deformación de seguridad estructural. Igualmente, ante la incidencia de carga cíclica no se deberá presentar degradación notable de la rigidez o adelgazamiento de la curva histerética de respuesta (pinching).

6.2 Procedimiento de diseño ante fuerza cortante.

Por normatividad de diseño, la resistencia esperada ante cortante de la unión V_{ju}, deberá ser mayor que la fuerza cortante obtenida del estado de esfuerzos empleado en el diseño para aseguramiento de la formación del mecanismo de fluencia V_j.

La resistencia a cortante de la unión se obtiene con la expresión (31)

$$V_{ju} = \kappa\ \sigma_B\ b_j\ D_j \qquad (31)$$

donde, κ coeficiente que depende de la configuración de la unión según la dirección de la carga incidente considerada, para unión interna con forma de + se toma 0.30, para unión exterior con forma de T o L se toma 0.18, D_j: peralte de la columna, o bien la distancia entre el paño de columna y el punto de doblez del acero de refuerzo longitudinal de la viga; b_j ancho efectivo de la unión a calcular con la expresión (32)

$$b_j = b_b + b_{a1} + b_{a2} \qquad (32)$$

donde, b_b: ancho de la viga; b_{ai} el menor valor de $b_i/2$ y $D/4$; b_i: distancia entre las caras laterales de viga y las caras laterales de la columna; D: peralte de la columna (para mayor claridad de la simbología consultar la Fig.9).

El refuerzo lateral p_{jh} en la unión deberá cumplir con los siguientes aspectos: el porcentaje de acero de refuerzo lateral deberá ser mayor a 0.002, exceptuando que se cumpla con la expresión (33)

$$p_{jh} \geq 0.003\ V_j / V_{ju} \qquad (33)$$

6.3 Anclaje del acero de refuerzo de columna y/o viga.

En extremos de vigas donde se proyecta la formación de articulaciones plásticas, se propone que el acero de refuerzo longitudinal de las mismas pase a través del núcleo de la columna, o en su defecto se ancle en el núcleo mismo. Para valuar la longitud de anclaje de los aceros de refuerzo tanto de vigas, como de columnas, en la unión, se considerará que dicha longitud inicia en las caras de cada elemento. El anclaje del refuerzo de viga en el núcleo de la columna se hará con un doblez de 90 grados, ubicando este doblez posterior al eje de columna

7. COMENTARIO FINAL.

El trabajo presentado aquí consiste en una traducción resumida de la "Guía para Diseño Antisísmico de Estructuras de Concreto Reforzado Empleando el Concepto de Resistencia Ultima"(4), que como su nombre lo indica no son normativas, ni reglamentarias, pero que contribuyen a disipar dudas y mostrar caminos para lograr diseños lógicos, razonados y económicos.

Es claro que aún permanecen muchos aspectos inconclusos, como son entre otros la determinación del sismo de diseño, la relación entre la cantidad de deformación plástica y las características del sismo de diseño, la cuantificación del factor de seguridad ante la incidencia de un sismo dado, la posibilidad de extender una guía o criterios de diseño como lo aquí presentado a estructuras irregulares o especiales. Sin embargo, es loable y de mucha utilidad la publicación de documentación como la mostrada en la referencia [4], ya que es producto de años de intensa investigación en el área de concreto reforzado

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[3] Park, R., Priestley, N. and Gill W., "Ductility of square-confined concrete columns", Proc. ASCE, Vol.108, No.ST4, April, 1982.
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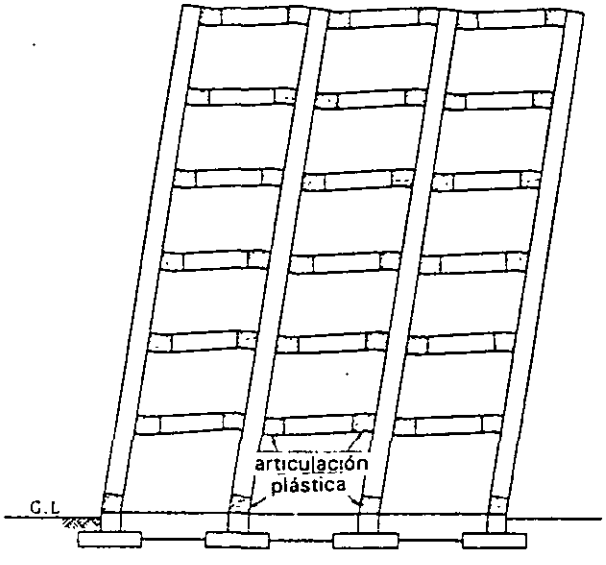

Fig.1 Mecanismo de fluencia de la estructura en base a plastificación en vigas

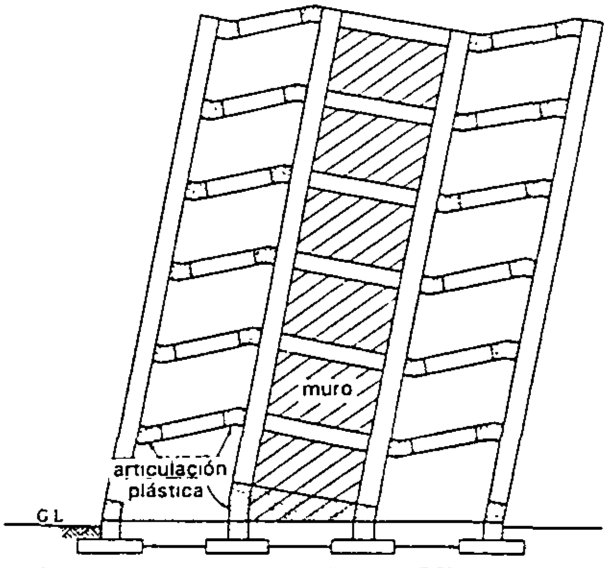

Fig.2 Mecanismo de fluencia de la estructura en base a plastificación de la base del muro

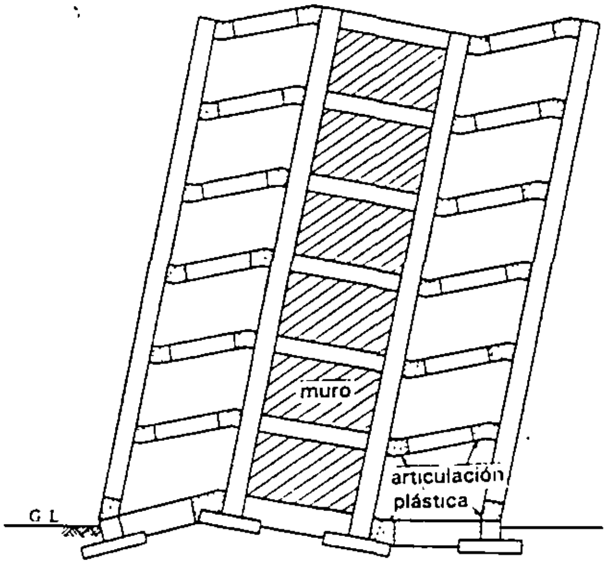

Fig.3 Mecanismo de fluencia de la estructura en base a plastificación de la contratrabe de cimentación

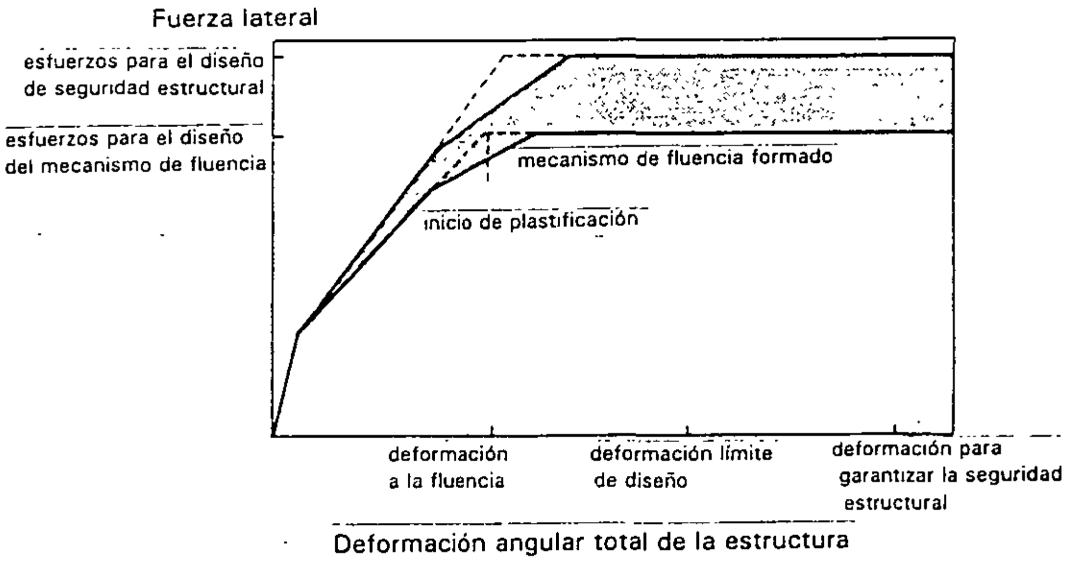

Fig.4.a Representación de los esfuerzos y deformaciones de diseño

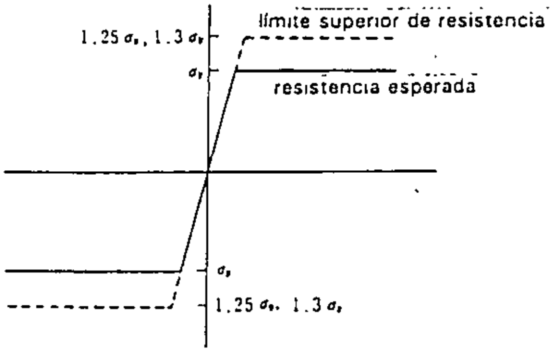

Fig.4.b Hipótesis en la relación esfuerzo-deformación del acero

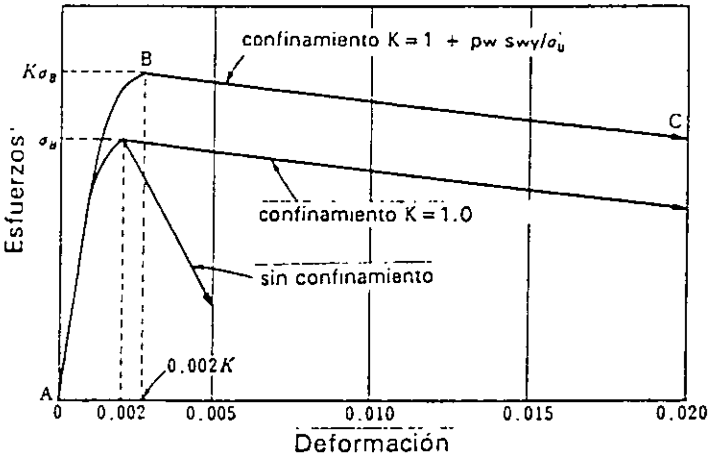

Fig.5 Relación esfuerzo-deformación del concreto confinado [3]

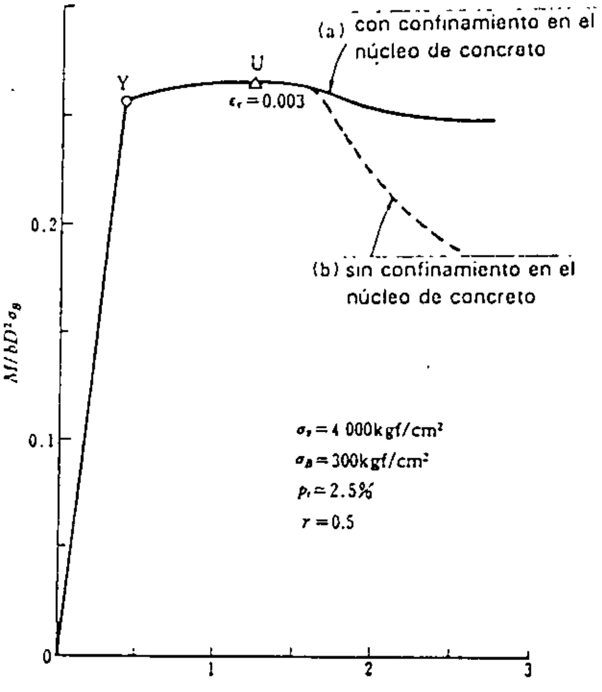

Fig.6 Relación momento - curvatura (calculado)

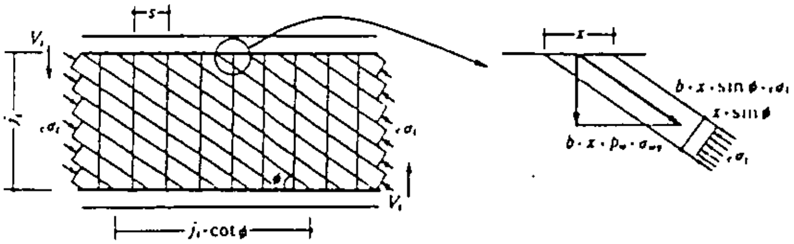

Fig.7.a Equilibrio de fuerzas para el mecanismo de armadura

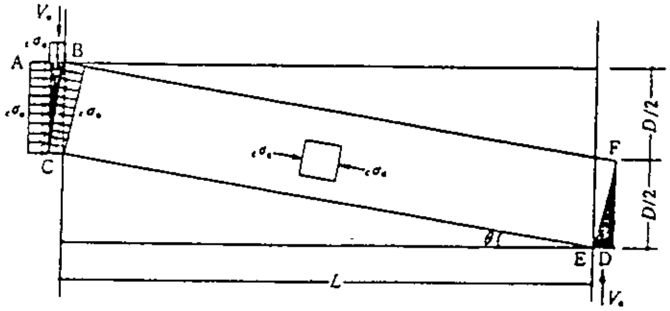

Fig.7.b Equilibrio de fuerzas para el mecanismo de arco

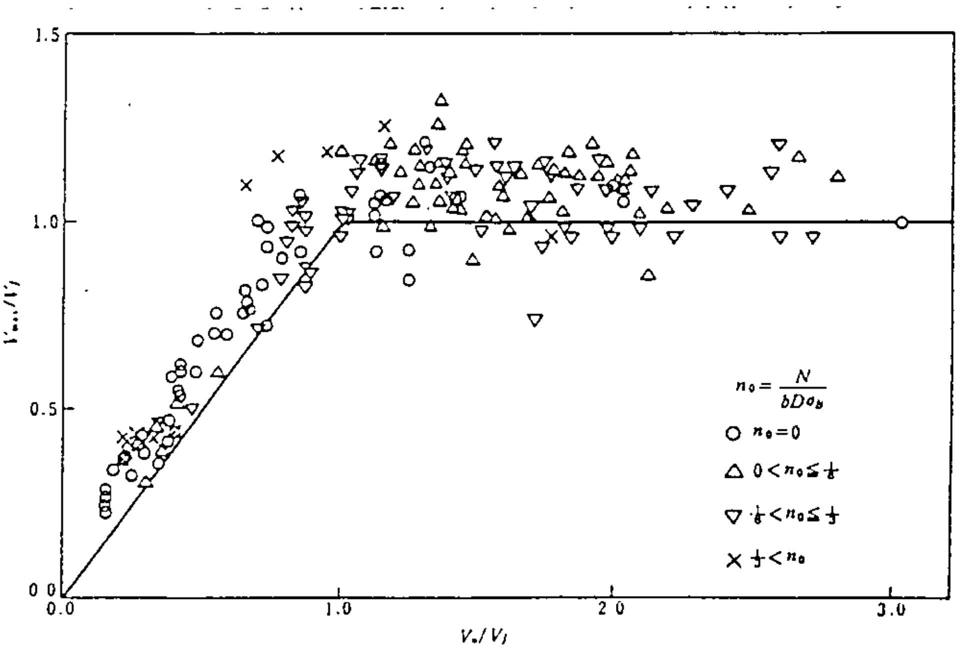

Fig.8 Comparación de valores de resistencia al cortante calculados con las expresiones expuestas, y resultados experimentales

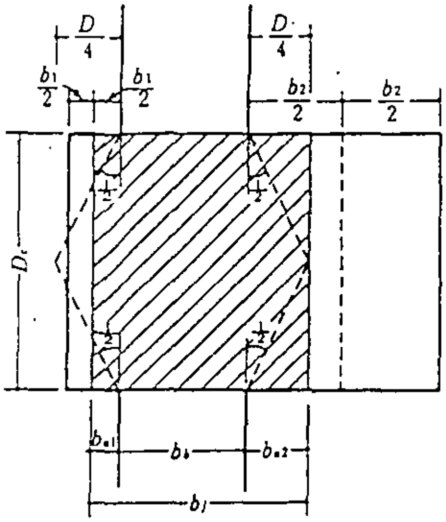

Fig.9 Ancho efectivo de la unión viga-columna

FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

XXVI CURSO INTERNACIONAL DE INGENIERIA SÍSMICA

MOUDLO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

LECCIONES DE SISMOS RECIENTES

EXPOSITOR: M. EN C. ENRIQUE DEL VALLE CALDERON
PALACIO DE MINERIA
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LECCIONES DE SISMOS RECIENTES

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Introducción

La ocurrencia de un movimiento sísmico intenso despierta siempre la atención de gran número de ingenieros, sismólogos y autoridades gubernamentales, pues mucho es aún lo que debemos aprender para poder reducir cada vez más los daños y pérdidas de vidas que producen dichos movimientos.

Las deficiencias de los reglamentos de construcción, que tienen siempre un cierto atraso en relación con los avances logrados en el campo de la ingeniería sísmica, las deficiencias en cálculo, en parte también por falta de actualización de los ingenieros; los defectos constructivos o el comportamiento indeseable de ciertos materiales de construcción; mala conservación o la acumulación de daños ocultos a traves de varios temblores, son espectacularmente expuestos a raíz de un sismo intenso. Dentro de ciertos intervalos, entre más antigua sea una construcción mayor será la probabilidad de que algunos de los conceptos antes mencionados se manifieste.

Unos de los problemas que suelen presentarse es la falta de costumbre de la gente o su incredulidad, cuando se dice que en un cierto lugar de la tierra el riesgo sísmico es elevado. Como es sabido, los períodos de recurrencia de los sismos intensos son, afortunadamente, largos, lo que hace que muchas veces las personas se olviden del riesgo que corren y empiecen a relajarse incluso los reglamentos o bien, no se preocupe nadie por establecerlos en caso de que no existan. Sólo cuando se presenta un movimiento intenso y provoca muchos daños, surge la necesidad de componer la situacion, pero esta efervescencia por desgracia es pasajera y al cabo de unos meses, todo se olvida y decae el interés.

Otras personas consideran tambien que sismos de mediana intensidad son suficientes para probar las bondades de ciertas prácticas de cálculo o constructivas, y animados por la ausencia de daños ante estos movimientos leves, insisten en su práctica, no siempre sana, a pesar de que temblores intensos han demostrado, quizá en otra parte del mundo, que no debe seguirse y estas experiencias son de su conocimiento

Poco a poco, a través de errores y fracasos, el hombre ha ido logrando el perfeccionamiento de los sistemas constructivos, así como el mejor conocimiento del comportamiento de los materiales al ser sometidos a los efectos de sismos intensos; sin embargo, aún falta mucho por hacer, sobre todo al nivel vivienda popular, en países poco desarrollados o en vías de desarrollo, donde la intervención del ingeniero no existe y siguen repitiéndose los errores, como por ejemplo, del uso de mampostería de adobe, sin reforzar, combinada con sistemas de techos pesados y que no contribuyen a la resistencia.

Sistemas estructurales

Para resistir fuerzas laterales provocadas por los sismos se disponen, básicamente, de sistemas estructurales a base de muros, sistemas estructurales a base de marcos rígidos constituídos por trabes y columnas unidas adecuadamente y sistemas estructurales constituídos por combinaciones de muros y marcos rígidos (ref 1)..

Los muros pueden ser de carga o rigidez y estar hechos de adobe, piedra, tabique hueco o macizo o bloques huecos de concreto o bien ser de concreto reforzado. En general son bastante eficientes para resistir fuerzas elevadas en su plano si se toman precauciones especiales para evitar problemas de falla frágil. La ductilidad que pueden alcanzar estos sistemas es variable, pero en general, es menor que la que se alcanza con otros sistemas.

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[1] Ingeniero Consultor

En ocasiones se usan grupos de muros unidos entre sí para formar tubos verticales, que pueden comportarse de manera muy eficiente para resistir los efectos sísmicos, con ductilidad adecuada

En muchos casos los muros no son considerados como elementos resistentes al momento de calcular la estructura: sin embargo, la falta de indicación de ésto en los planos constructivos, aunada a prácticas constructivas deficientes, muchas veces de buena fé, pero ignorantes del problema que puede ocasionarse, hace que se integren a los elementos que resistirán los efectos sísmicos, provocando serios problemas, como se verá más adelante

Los sistemas estructurales a base de marcos rígidos son bastante empleados en la construcción de edificios de uso general, en los que se desconoce la distribución de los espacios, durante la etapa de cálculo y se desea dar amplia libertad de uso. Se conocen también como estructuras esqueléticas y se construyen principalmente de concreto reforzado o de acero estructural aunque también suele usarse la madera en ciertos casos.

Este tipo de estructura puede desarrollar una buena ductilidad bajo la acción de los efectos sísmicos . Su elevada hiperestaticidad y el comportamiento más allá del límite elástico, permiten la redistribución de efectos sísmicos y los hace especialmente adecuados para resistir fuerzas laterales en edificios altos; sin embargo, es frecuente que su comportamiento se vea obstaculizado por elementos no estructurales, lo que conduce a problemas de mayor o menor importancia.

Las deformaciones laterales de este tipo de estructuras son mayores, en general, que las de sistemas a base de muros, y deben dejarse las holguras constructivas necesarias para que esas deformaciones puedan tener lugar, previendo las conexiones adecuadas de instalaciones, fachadas, muros divisorios, etc. En algunas ocasiones se emplean contravientos diagonales o muros de rigidez con objeto de reducir las deformaciones.

El empleo cada vez mas frecuente de computadoras digitales para el análisis de este tipo de sistemas ha ido eliminando los problemas asociados a subestimaciones o sobre estimaciones de sus propiedades elástico-geométricas por el empleo de métodos aproximados de análisis, sin verificar si se cumplen las restricciones de dichos métodos. Puede citarse como ejemplo la determinación de rigideces de entrepiso, y por consiguiente, de las deformaciones laterales que sufrirá la estructura, en marcos construídos por columnas relativamente robustas en comparación con las trabes (ref 2).

Es bastante frecuente en nuestros días la combinación de sistemas a base de muros y a base de marcos. El problema fundamental de esta combinación es la determinación de la compatibilidad de deformaciones de ambos sistemas al estar sometidos a fuerzas horizontales, ya que su comportamiento aislado es completamente diferente. Puede ser muy eficiente esta combinación en edificios de gran altura. El empleo de computadoras digitales en el análisis es imprescindible para lograr una predicción adecuada del comportamiento de la estructura.

La estructuración que se adopte es fundamental en el éxito o fracaso de un edificio. El ingeniero estructurista no puede lograr que una forma estructural pobre, tal vez por causa de un capricho arquitectónico, se comporte satisfactoriamente en un temblor. Existe una serie de recomendaciones de tipo general (ref 3), que es conveniente seguir para lograr buenos resultados Aún cuando no existe una forma universal para un tipo particular de estructura, esta debe ser, siempre que sea posible: simple; simétrica,no demasiado alargada ni en planta ni en elevación; ser uniforme y tener su resistencia distribuída en forma uniforme, sin cambios bruscos; tener miembros horizontales en los que se formen articulaciones plásticas, antes que en los miembros verticales y tener su rigidez en relación con las propiedades del subsuelo.

Esta última condición no se ha respetado en muchas ocasiones y ha sido causa de problemas importantes. En general, se sabe que una estructura flexible se comporta mejor cuando está desplantada en un suelo rígido y una rígida cuando lo está en suelo blando. Aunque en esta definición quedan demasiado vagos los terminos de rigidez de estructuras y suelos, lo importante es que haya bastante diferencia, de ser posible entre los períodos dominantes propios del terreno y de la estructura. Esto fue claramente demostrado en los sismos de Septiembre de 1985 en la Ciudad de México,(ref 18).

Elementos no estructurales

Se consideran como elementos "no estructurales" aquellos que no contribuyen, teóricamente, a la resistencia de la estructura al ser sometida a los efectos sísmicos, tales como muros divisorios o de colindancias , fachadas, plafones, instalaciones hidráulicas, eléctricas, o de otro tipo, tanques, antenas, etc.

Los principales problemas son causados por la unión inadecuada de estos elementos a la estructura, provocando que, al deformarse ésta, se recargue con mayor o menor intensidad en aquellos. que al no estar diseñados para resistir los efectos del sismo, pueden sufrir daños considerables.

En muchos temblores recientes, las mayores pérdidas económicas han ocurrido en elementos no estructurales, sobre todo en muros divisorios, de colindancia o de fachadas, debido a su elevada rigidez (no siempre compatible con su resistencia) que impide la deformación de la estructura si no hay holguras constructivas adecuadas.

Es frecuente que la estructura también resienta daños importantes, pues no está diseñada para tomar los esfuerzos que le transmiten los muros.

Resulta pues sumamente importante definir claramente en los planos constructivos cuales son los elementos que forman parte integrante de la estructura y cuales son "no estructurales", indicando la forma en que deben colocarse, las holguras constructivas que deben dejarse, incluyendo los acabados y otras precauciones que se juzguen pertinentes.

Daños observados

A continuación se discutirán los principales tipos de daños en temblores recientes, tomando en cuenta los comentarios hechos con anterioridad.

Es necesario definir si los daños pueden poner en peligro la estabilidad de la estructura o son en elementos no estructurales, sin peligro de colapso, pero con costos de reposición elevados.

Los daños pueden consistir en:

- agrietamientos ligeros de acabados y muros no estructurales
- agrietamientos fuertes de acabados y muros no estructurales
- agrietamientos ligeros de muros estructurales
- agrietamientos severos de muros estructurales
- formación de articulaciones plásticas en columnas o trabes o fracturas importantes
- colapsos parciales de elementos no estructurales
- colapsos parciales de elementos estructurales
- colapsos totales
- pérdida de verticalidad de la estructura
- fallas de anclaje del refuerzo
- desconchamiento del recubrimiento
- pandeo local o generalizado
- rupturas de tuberías o ductos de instalaciones
- colapso de plafones
- golpeo contra construcciones vecinas por flexibilidad excesiva
- fractura de losas o escaleras.

Los informes del Instituto de Ingeniería No. 313 y 324 elaborados por el suscrito sobre los temblores de Managua el 23 de diciembre de 1972 y del ocurrido en una amplia región de México el 28 de agosto de 1973, ilustran la mayoría de los daños antes mencionados.

Se puede encontrar información adicional en numerosas publicaciones, algunas del mismo Instituto de Ingeniería de la UNAM, por ejemplo las referencias 4 a 6, o bien, descripciones de daños por temblor que han sido presentadas en los distintos congresos mundiales de ingeniería sísmica, referencias 7 a 16. El capítulo 9 de la referencia 17 ilustra el comportamiento de estructuras en los Estados Unidos a través de diversos temblores. Sobre el temblor de 1985 en la Ciudad de México se escribieron numerosos informes, ver por ejemplo la ref 18.

Muchos de los daños que se han presentado podrían haberse evitado tomando precauciones mínimas durante la construcción. En otros casos, la intensidad del movimiento rebasó las predicciones que tenían, o superó la capacidad estimada para las estructuras, obligadas en ambos casos a modificar los reglamentos de construcción.

Actualmente se han refinado bastante las técnicas para estimar la sísmicidad de un lugar. La determinación de la resistencia de las estructuras sometidas a sismos es también motivo de numerosas investigaciones.

Algunos comentarios sobre la reparación de estructuras dañadas

Después de cada temblor intenso, un buen número de estructuras quedan con daños estructurales más o menos severos y es necesario decidir si se reparan o se demuelen. En caso de repararlas, es preciso definir como debe llevarse a cabo la reparación.

No es fácil, de la simple observación de los daños, apreciar que tan afectada puede estar una estructura. Es poco también lo que se conoce en relación con la acumulación de daños por temblor a través de varios movimientos intensos.

La reparación de una estructura debe hacerse a partir de un análisis muy detallado de la misma, teniendo especial cuidado de no alterar localmente sus propiedades resistentes, pues temblores futuros se encargarán de poner en evidencia las fallas que han sido inadecuadamente reparadas. La reparación local de elementos resistentes, bastante frecuente, puede conducir a un aumento en la rigidez del elemento reparado por lo que, en otro sismo, tomará mayor fuerza sísmica y puede volver a fallar, quizá con resultados peores que en la primera ocasión. Es muy frecuente que sea necesario reforzar elementos sanos con objeto de repartir las cargas sísmicas en una forma más adecuada. En ocasiones es conveniente poner una nueva estructura, quizá metálica, adosada a la dañada, más rígida que ésta, para absorber los efectos sísmicos en su totalidad cuidando que los sistemas de piso sean capaces de transmitir las fuerzas sísmicas adecuadamente.

En muchas construcciones de mampostería, el simple resane de los agrietamientos, sin estudiar por qué se agrietaron y qué puede pasar en temblores futuros, es muy peligroso, pues la estructura puede haber perdido gran parte de su capacidad a fuerzas laterales y sufrir colapsos importantes en temblores futuros. En ocasiones es mejor sustituir el elemento de mampostería dañado o reforzarlo adecuadamente. Se ha visto que un aplanado reforzado con malla puede restituir eficientemente la resistencia; sin embargo, será necesario estudiar el comportamiento de conjunto de la estructura, para decidir si sólo se refuerzan los elementos dañados o también se refuerzan otros elementos, aparentemente sanos, pero que requieren ser reforzados para lograr un trabajo de conjunto eficiente.

Es muy frecuente que ciertas deficiencias en sistemas constructivos o estructurales hayan sido puestas en evidencia en un lugar, y que esos mismos defectos sean comunes en otro lugar con sismicidad semejante, pero en el cual, hace tiempo que no han ocurrido temblores.

Lo normal es, que a pesar de saber que puede haber serios daños en el segundo lugar cuando ocurra un sismo, no se haga nada para prevenirlos. Ciertamente es difícil, como ya se dijo antes, convencer a la gente del riesgo en que se encuentra, y tal vez haya que esperar a que ocurran los daños, para que se tomen cartas en el asunto. Evidentemente, la divulgación de este problema a nivel de autoridades gubernamentales, compañías de seguros, ingenieros estructuristas, arquitectos, etc. ayudará en la solución de este dilema.

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EFECTO DE LOS SISMOS EN LAS CONSTRUCCIONES

Enrique del Valle C.[1]

Características dinámicas.

El efecto de los sismos sobre las estructuras depende de las características dinámicas tanto de la estructura como del movimiento. El problema es sumamente complejo, pues las características dinámicas del movimiento son variables tanto durante un mismo temblor, como de uno a otro temblor, dependiendo de la distancia epicentral, profundidad focal y magnitud del sismo, así como del tipo de terreno en que estén desplantadas las estructuras.

Las características de interés del movimiento son la duración, la amplitud y la frecuencia, refiriéndose la amplitud a los máximos valores que se alcanzan durante el sismo, ya sea de desplazamiento, velocidad o aceleración del suelo y la frecuencia al número de ciclos de oscilación del movimiento por unidad de tiempo. En general, en terrenos firmes la frecuencia es más alta que en terrenos blandos, lo que indica que el número de ciclos de oscilación del terreno por unidad de tiempo es mayor, sintiéndose el movimiento mucho más violento y rápido que en los terrenos blandos, donde es más lento; los desplazamientos y la duración total suelen ser mucho mayores en el terreno blando.

Por otro lado, las características dinámicas de las estructuras no son fáciles de estimar correctamente, debido a las incertidumbres existentes en la determinación de las propiedades elástico-geométricas de los elementos que conforman las estructuras, a la variación de las propiedades al presentarse comportamiento inelástico, así como a incertidumbres en cuanto a la colaboración a la resistencia y rigidez de elementos no estructurales, que suelen participar en la respuesta sísmica debido a que es difícil desligarlos adecuadamente de la estructura, también es poco frecuente incluir la participación de la cimentación y del suelo circundante en la determinación de las propiedades dinámicas de un edificio.

Se define como rigidez lateral o de entrepiso a la oposición de la estructura a ser deformada entre un nivel y otro por las cargas horizontales aplicadas en cada nivel. Puede hablarse también de rigidez angular, que será la oposición de un nudo de una estructura o del extremo de un elemento estructural a girar al ser sometido a un momento flexionante; o de rigidez lineal, que será la oposición al desplazamiento relativo de un extremo de un miembro estructural con respecto a su otro extremo (fig. 1).

La rigidez, tanto de entrepiso como angular o lineal, depende del tamaño de la sección transversal de los elementos estructurales, con lo que se calculan las propiedades geométricas: áreas y momentos de inercia, de su longitud, de la forma en que están conectados a otros elementos y del material con que están hechos, lo que define las propiedades elásticas como módulo de elasticidad, módulo de Poisson y módulo de cortante.

Es una propiedad diferente a la resistencia, aunque a veces se confunde con ella. Hay elementos estructurales en que existe compatibilidad entre resistencia y rigidez, pero hay otros en que la rigidez es mucho mayor que la resistencia, como es el caso de los muros de mampostería, lo que complica el problema de análisis de las estructuras en que existen elementos de este tipo. Asimismo, las propiedades elásticas del acero están más definidas que las del concreto reforzado o de la mampostería.

Cuando el nivel de esfuerzos a que están trabajando los materiales es bajo, su comportamiento puede ser cercano al elástico, esto es, habrá proporcionalidad entre esfuerzos y deformaciones, correspondiendo una deformación del doble para esfuerzos dos veces mayores; pero, a medida que los esfuerzos crecen, el comportamiento deja de ser

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elástico, alcanzándose lo que se conoce como comportamiento no lineal o inelástico, en el cual, al duplicar el esfuerzo, la deformación es mucho mayor que el doble a que se hizo mención antes (fig.2).

Debido a lo anterior, en general se elaboran modelos matemáticos elásticos muy simplificados de las estructuras, pues, aún con ayuda de las computadoras, el problema dista de ser manejable. Entre las características más importantes que pueden obtenerse de los modelos están los periodos de oscilación de cada uno de los distintos modos en que pueden vibrar y las formas de estos modos, entendiéndose por periodo el tiempo que tarda en ocurrir una oscilación completa (fig.3).

Otras características importantes de las que depende la respuesta de la estructura son el amortiguamiento y la ductilidad que pueden desarrollarse El amortiguamiento es una propiedad intrínseca de los materiales empleados, pero depende también de la forma en que se conecten los miembros estructurales y los no estructurales: Valores de amortiguamiento relativamente pequeños reducen considerablemente la respuesta sísmica de las estructuras.

Se conoce como amortiguamiento crítico el que tiene una estructura cuando, al separarla de su posición y soltarla no oscila sino que regresa a la posición de equilibrio; las estructuras suelen tener amortiguamiento del orden de 3 a 10% del crítico, siendo menor el de las estructuras metálicas, soldadas y sin recubrir, y mayor el de las estructuras de mampostería, con gran número de juntas. Puede aumentar algo al someter a las estructuras a grandes deformaciones. También puede aumentarse colocando amortiguadores de diseño especial, que están empezando a desarrollarse.

La ductilidad de las estructuras es la propiedad de soportar grandes deformaciones inelásticas sin fallar ni reducir su capacidad de carga. Depende en gran medida de los materiales empleados y de los cuidados que se tienen al diseñarlas. Es una propiedad muy deseable en las estructuras situadas en zonas sísmicas, pues por lo general no es aconsejable diseñar las estructuras sometidas a estas acciones sobre la base de un comportamiento elástico, ya que sería antieconómico debido a la escasa probabilidad de que ocurra el sismo de diseño durante la vida útil de la estructura, además de que es muy difícil saber cuál será la mayor excitación sísmica que puede ocurrir, pues la historia con que se cuenta aún en países habitados hace muchos siglos no es suficiente para ello.

Espectros de respuesta.

Conocidos los acelerogramas de temblores intensos es posible estimar la respuesta de modelos simples en función del tiempo y, por consiguiente, la respuesta máxima que puede ocurrir en un instante dado. Esto puede hacerse considerando que el comportamiento de la estructura será elástico en todo el evento o bien que se incursionará en el intervalo de comportamiento inelástico a partir de un cierto valor de respuesta.

La gráfica que relaciona las respuestas máximas de distintas estructuras sometidas a una misma excitación con sus periodos de oscilación recibe el nombre de espectro de respuesta (fig. 4). Según el tipo de comportamiento que se haya considerado se tendrán espectros de respuesta elásticos o espectros de respuesta inelásticos.

Normalmente estos espectros se obtienen suponiendo que las estructuras tienen distintos valores del porcentaje de amortiguamiento crítico, pues, como se dijo antes, un pequeño valor de éste es suficiente para reducir considerablemente la respuesta . Los valores empleados normalmente en cálculos de este tipo son 0, 2, 5, 10 y 20% del amortiguamiento crítico (fig. 5).

El tipo de terreno en que se haya obtenido el acelerograma es muy importante, pues las características dinámicas de la excitación varían en función de esto. Como ya se indicó, en suelos firmes las vibraciones son rápidas, mientras que en suelos blandos las oscilaciones son de menor frecuencia , esto es, su periodo es relativamente más largo. Esto modifica la forma de los espectros de respuesta.

Las respuestas que suelen calcularse son desplazamientos, velocidades o aceleraciones, pues a partir de ellas se puede calcular cualquier efecto que se desee conocer en la estructura, como por ejemplo momentos de volteo en la base, fuerzas cortantes en cualquier nivel, esfuerzos en alguna sección, etc.

Normalmente los acelerogramas tienen periodos que varían dentro de una banda de valores relativamente ancha; sin embargo, en cierto tipo de suelo y bajo condiciones especiales puede haber algún periodo dominante en particular, como ocurrió en el sismo del 19 de septiembre de 1985, en el acelerograma obtenido en el centro SCOP, en el que se observa un periodo muy definido de dos segundos de duración. Esta situación conduce al problema dinámico conocido como resonancia, que consiste en una amplificación excesiva de la respuestra de aquellas estructuras que tienen algún periodo de oscilación muy parecido al de la excitación, lo que puede llevarlas al colapso total, sobre todo cuando la duración del evento es grande.

Es muy fácil demostrar por medio de una mesa vibratoria, en la que se coloquen modelos de las estructuras con diferentes periodos de oscilación, que la respuesta de uno de ellos se puede amplificar considerablemente moviendo la mesa con un periodo igual al de ese modelo, observándose que los otros no sufren mayores oscilaciones. Al cambiar el periodo de la oscilación, se excitará algún otro modelo, y así sucesivamente (fig. 6).

Criterios de Diseño Sísmico.

Los criterios de diseño por sismo (filosofía del diseño sísmico) adoptados por la mayor parte de los reglamentos de construcción de los países que tienen problemas sísmicos establecen la necesidad de diseñar las estructuras para resistir, sin daños, sismos de baja intensidad, de ocurrencia relativamente frecuente, prevenir daños estructurales y minimizar daños no estructurales que pudieran ocurrir en sacudidas ocasionales de intensidad media y evitar el colapso o daños serios en caso de sacudidas del terreno de intensidad extrema, pero de probabilidad de ocurrencia muy baja, permitiendo daños no estructurales y aún estructurales en este caso (ref. 1). Esto obedece, como se indico anteriormente, a motivos económicos, considerando muy baja la probabilidad de que se presente un sismo muy intenso, igual o mayor que el propuesto para diseño, durante la vida útil de la estructura.

Sin embargo, se reconoce que los datos estadísticos actuales no permiten desarrollar correctamente estos criterios de diseño, lo cual fue claramente demostrado con el sismo de septiembre de 1985 que rebasó ampliamente las previsiones que se tenían para diseño.

En los criterios en vigor se aprovecha la propiedad de ductilidad de las estructuras, la que también es útil para compensar la subestimación del máximo sismo que puede presentarse en un lugar, por falta de información adecuada, como ocurrió en el sismo de septiembre de 1985.

En los reglamentos se proponen usualmente valores máximos para diseño, estimados con base en la información estadística de que se disponga, considerando que las estructuras tienen comportamiento elástico. Suelen proponerse espectros para diseño obtenidos como una envolvente de espectros de respuesta de distintos temblores, escalados a una cierta intensidad. Para el cálculo de las fuerzas equivalentes al sismo se permite reducir por ductilidad los valores máximos antes mencionados, dependiendo del tipo de estructura, ya sea de marcos rígidos, muros de carga y rigidez, o combinación de estos sistemas, de la regularidad de la estructura, de los materiales con que está hecha y de los cuidados que se tengan en el detallado y construcción.

Recomendaciones sobre estructuración.

Con base en la experiencia obtenida en muchos temblores ocurridos en distintas partes del mundo se ha elaborado una serie de recomendaciones sobre estructuración, para lograr un mejor comportamiento sísmico, entre las más importantes están las siguientes (refs. 2, 3, 4, 5).

A) Poco peso.

B) Sencillez, simetría y regularidad tanto en planta como en elevación.

C) Plantas poco alargadas.

D) Uniformidad en la distribución de resistencia, rigidez y ductilidad.

E) Hiperestaticidad y líneas escalonadas de defensa estructural.

F) Formación de articulaciones plásticas en elementos horizontales antes que en los verticales.

G) Propiedades dinámicas adecuadas al terreno en que se desplantará la estructura.

H) Congruencia entre lo proyectado y lo construido.

Se recomienda que las estructuras sean ligeras pues las fuerzas debidas al sismo surgen como consecuencia de la inercia de las masas a desplazarse, por lo que, entre menos pesen, menores serán los efectos de los sismos en ellas. Conviene también que sean sencillas, para que los modelos matemáticos sean realistas, pues una estructura muy compleja, mezclando distintos tipos de sistemas estructurales y materiales, no es fácil de modelar; que sean simétricas para reducir efectos de torsión, por lo que se debe evitar las plantas en forma de L, T, C, y triangulares (fig 7); que no sean muy alargadas ni en planta, ni en elevación: en planta, para reducir la posibilidad de que el movimiento de un extremo del edificio sea diferente al del otro extremo, lo que causaría efectos usualmente no previstos; en elevación, para reducir los efectos de volteo, que encarecen considerablemente las cimentaciones. Se deben evitar remetimientos en elevación (fig. 8), pues los cambios bruscos en masa o rigidez propician amplificaciones dinámicas importantes, que suelen provocar daños graves. Lo mismo puede decirse con respecto a cambios en la forma de la planta, debiendo limitarse la extensión de apéndices que sobresalgan, como en el caso de formas simétricas en cruz o en H.

Conviene que la resistencia y la rigidez de la estructura estén repartidas uniformemente, sin concentrarse en unos cuantos elementos resistentes, o con variaciones grandes en los claros entre columnas o en las dimensiones de las trabes y de las columnas. Entre mayor hiperestaticidad tiene una estructura, es mayor el número de secciones estructurales que deben fallar antes de que la estructura colapse, asimismo, si se planea que haya elementos que fallen antes que otros, se puede dar la posibilidad de evitar daños grandes a toda la estructura. Estos elementos deben colocarse adecuadamente para que su reparación sea sencilla. El problema de satisfacer esta condición es que se requiere analizar varias etapas del comportamiento, para verificar que los elementos estructurales que van quedando son capaces de soportar el sismo sin colapsar, lo que encarece y complica el cálculo de la estructura.

Se debe buscar una estructuración a base de columnas fuertes-vigas débiles, para propiciar la formación de articulaciones plásticas en las vigas al excederse la resistencia suministrada, ya sea porque se está aprovechando la ductilidad o porque, además de eso, el sismo excede las previsiones de diseño. Al proceder así se logran mecanismos que pueden evitar más fácilmente el colapso de la estructura, pues la demanda de ductilidad local en las trabes de todos los entrepisos reparte mejor los efectos del sismo que cuando la demanda de ductilidad se concentra en las columnas de un sólo entrepiso (fig. 9). Por otro lado, el comportamiento dúctil de elementos estructurales sujetos a flexión pura, como en el caso de las trabes, es mucho mejor que el de elementos a flexocompresión, que es el caso de columnas (figs. 10 y 11).

Se recomienda también que se busque que las propiedades dinámicas de la estructura sean congruentes con las del suelo en que está desplantada; en general se dice que en suelos firmes se comportan mejor las estructuras flexibles y en suelos blandos las estructuras rígidas. Lo que trata de evitarse con esta recomendación es la posible resonancia por coincidencia de las propiedades dinámicas de la estructura y del suelo, como la observada el 19 de septiembre.

Finalmente, es recomendable también que lo que se construye sea congruente con lo que proyecta; en muchas ocasiones, al proyectar una estructura se decide no aprovechar la colaboración de muros de relleno, debido a la posibilidad de que sean eliminados para dejar libertad en la distribución de espacios en el proyecto arquitectónico de los distintos niveles; sin embargo, suele no detallarse adecuadamente la forma en que estos muros deben construirse, desligados de la estructura, para permitir que ésta se deforme sin recargarse en ellos, pues si lo hace les transmitirá buena parte de la fuerza sísmica que debía absorber, debido a que los muros, sobre todo cuando son de mampostería, tienen una rigidez intrínseca bastante alta en su plano, aunque su resistencia no sea compatible con esa rigidez, como se mencionó antes. Si los muros de relleno colaboran con la estructura para resistir los efectos sísmicos sin haber sido calculados para absorber la fuerza que les corresponde en función de su rigidez, el comportamiento de la estructura será muy distinto al supuesto en el proyecto estructural, pudiendo presentarse muchos daños.

En algunos casos la colaboración de los muros no estructurales evita el colapso de estructuras subdiseñadas, si su colocación es relativamente simétrica y tiene continuidad de un piso a otro. Pero cuando su colocación es asimétrica, como ocurre en los muros de colindancia de edificios en esquina o cuando son discontinuos, como ocurre en edificios de departamentos en que la planta baja o algunos otros niveles no tienen muros porque se destinan a estacionamiento o comercios, la colaboración de los muros de relleno pueden ser causa de daños muy graves o aún de colapso total de la estructura, al propiciar efectos torsionantes importantes en el primer caso o una condición de piso "suave" en el segundo.

El cambio de cargas con respecto al proyecto suele ser también causa de daños importantes en las estructuras. Usualmente un edificio diseñado para resistir el efecto combinado de cargas verticales y cargas de sismo puede soportar sin problemas sobrecargas verticales importantes mientras no tiemble, pero, si existe sobrecarga al momento de un sismo, los efectos de éste se verán doblemente amplificados, por lo que pueden ocurrir daños importantes o colapsos parciales o totales.

Ingeniería Sísmica. Métodos de Análisis por Sismo.

La ingeniería sísmica empezó a desarrollarse y a proponer recomendaciones para diseño sísmico hace unos setenta años, después del temblor de 1923 en Japón, donde se vió que algunas estructuras diseñadas con ciertos principios habían resistido el sismo satisfactoriamente. Al principio los avances fueron relativamente lentos pero poco a poco se ha logrado mejorar los criterios de diseño a nivel internacional, sobre todo a raíz de la creación de la Asociación Internacional de Ingeniería Sísmica, que organiza cada cuatro años aproximadamente, a partir de 1956, congresos mundiales de ingeniería sísmica, donde los ingenieros de todo el mundo tienen oportunidad de intercambiar ideas y experiencias. Esos congresos mundiales se han celebrado en Estados Unidos de América en 1956, Japón en 1960, Nueva Zelanda en 1965, Chile en 1969, Italia en 1973, la India en 1977, Turquía en 1980 y nuevamente en los Estados Unidos de América en 1984, en Japón en 1988 y en Madrid, España en 1992, cada vez con mayor cantidad de ponencias y participantes. Además de la memorias de los distintos congresos, la Asociación ha editado también una publicación que resume los distintos reglamentos de diseño sísmico de los países miembros, México entre ellos, donde se puede ver el grado de desarrollo alcanzado en cada país (ref. 6.).

En nuestro país el interés por la ingeniería sísmica se desarrolló de manera importante después del temblor del 28 de julio de 1957, que causó grandes destrozos en la Ciudad de México, dando lugar a la revisión del reglamento de construcción existente y propiciando la investigación e instrumentación sísmica.

En general, como se dijo antes, el problema dinámico que originan los temblores en estructuras es sumamente complejo y difícil de representar analíticamente, por lo que en los reglamentos se recomiendan usualmente métodos de análisis relativamente simples, que tratan de representar los efectos del sismo a través de fuerzas horizontales aplicadas en los distintos niveles de un edificio, evaluadas ya sea por un método estático o bien por métodos dinámicos, que tratan de ser más precisos.

Métodos estáticos. Los métodos más comunmente empleados en el análisis sísmico son los llamados estáticos, las fuerzas equivalentes al efecto sísmico se valúan considerando una aceleración en el nivel inferior (coeficiente sísmico) reducida por ductilidad, que, al multiplicar por el peso total del edificio, da como resultado una fuerza cortante en la base del mismo. Esa fuerza total se reparte en los distintos niveles en función de su peso y ubicación con respecto al nivel inferior, tratando de obtener una envolvente del comportamiento de la estructura a la excitación en su base. En estos métodos sólo se requiere conocer la ubicación y destino de la estructura para asignarle un coeficiente sísmico adecuado, que tome en cuenta el tipo de terreno en que se desplantará y el tipo de ocupación que tendrá; el sistema estructural y los materiales que se emplearán, para estimar la ductilidad que podrá desarrollarse y reducir las fuerzas en función de ella; los pesos de los distintos niveles, la ubicación de su centroide y sus alturas respecto a la base, para evaluar el cortante basal, repartirlo a los distintos niveles y obtener momentos de volteo y efectos torsionales. Se requiere también conocer las rigideces de los distintos elementos resistentes, para calcular los desplazamientos máximos probables y estimar los efectos que el sismo ocasionará en cada elemento estructural: trabes, columnas, losas y muros.

Para las estructuras más comunes, que son construcciones de muros de carga de mampostería de uno a tres niveles, existen métodos estáticos simplificados, que pueden emplearse si se cumple con una serie de requisitos; con estos métodos sólo se necesita revisar si la capacidad resistente de la mampostería a fuerza cortante es suficiente, sin tener que calcular desplazamientos laterales, momentos de volteo y efectos torsionales.

Métodos Dinámicos. Los métodos dinámicos se aplican en la determinación de los efectos sísmicos en edificios altos, cuya respuesta puede complicarse por la participación de modos superiores de vibrar, así como por las posibles variaciones de masa y rigidez en elevación. Es necesario elaborar modelos matemáticos más o menos refinados de la estructura, tomando en cuenta en ocasiones su carácter tridimensional, para calcular las formas en que puede oscilar y los periodos correspondientes, empleando computadoras. Con estos datos se estima, mediante un espectro de diseño, la máxima respuesta de cada uno de los modos de vibrar y se combinan para obtener fuerzas máximas probables que actuarán sobre la estructura, debido al sismo de diseño.

Cabe mencionar que, para el cálculo de desplazamientos, no se permite reducir las fuerzas, pues se estima que los desplazamientos elásticos bajo las fuerzas máximas son aproximadamente iguales a los calculados con las fuerzas reducidas, multiplicados por el factor de reducción por ductilidad, ya que al reducir las fuerzas se está permitiendo que la estructura se deforme inelásticamente.

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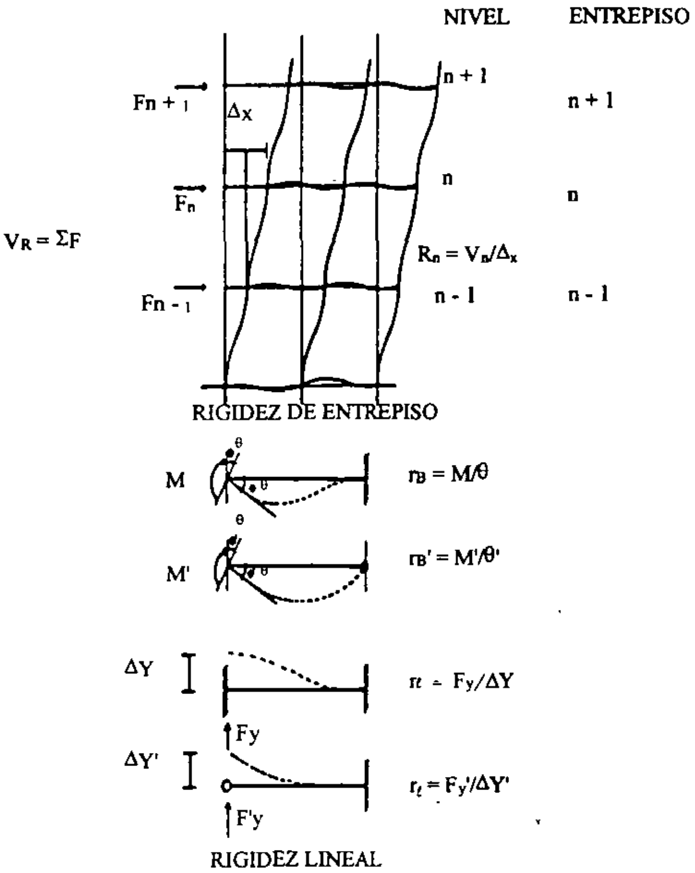


Fig. 1. Rigidez de Entrepiso, Angular o Lineal

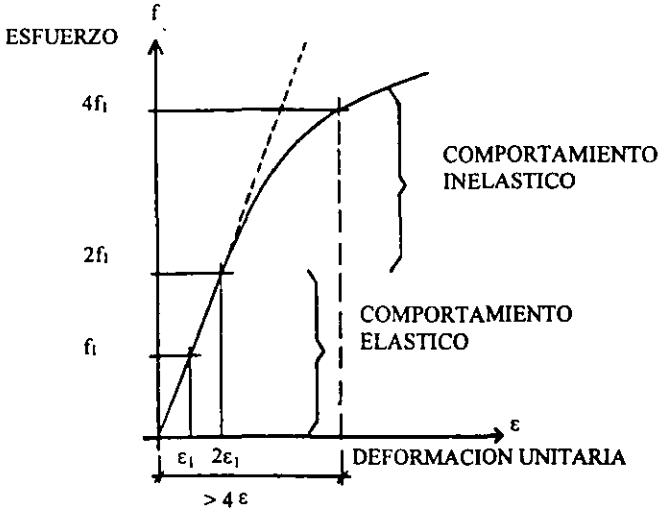


Fig. 2. Comportamiento Elástico e Inelástico de los Materiales.

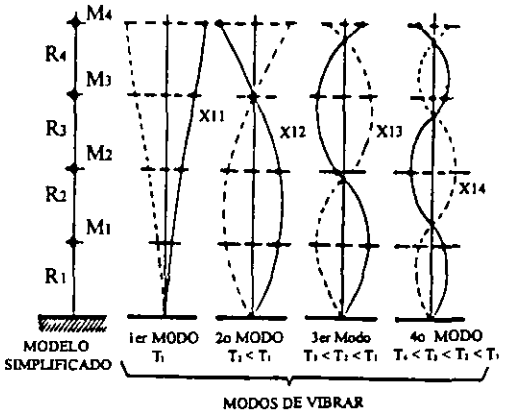


Fig. 3. Modelo Simplificado de un Edificio y Modos de Vibrar

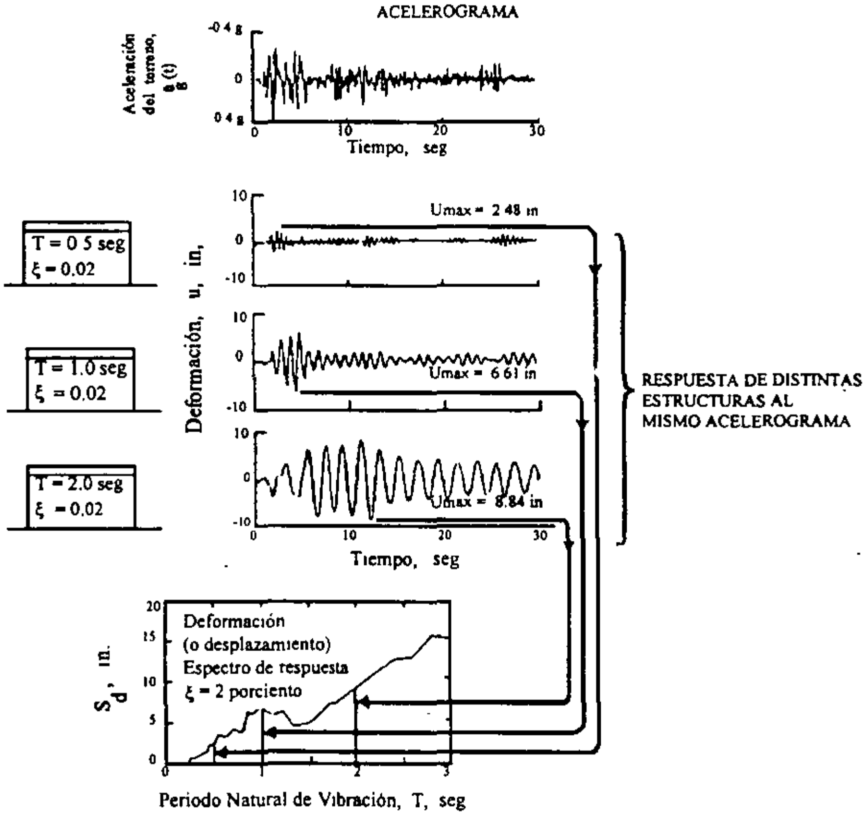


Fig.4. Construcción de un Espectro de Respuesta

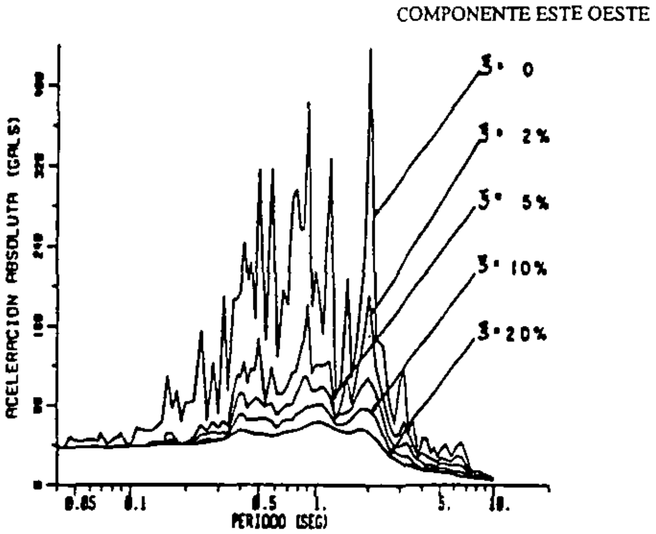


Fig.5. Espectros de Respuesta para la Componente N.S. Reg. de C.U.

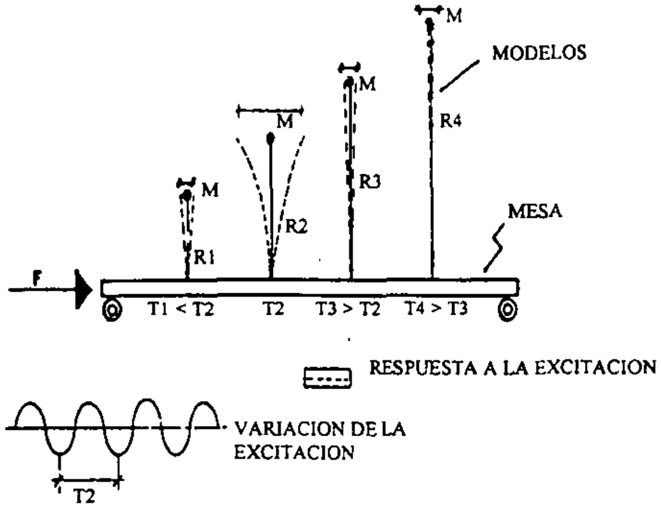


Fig.6. Demostración de la Resonancia

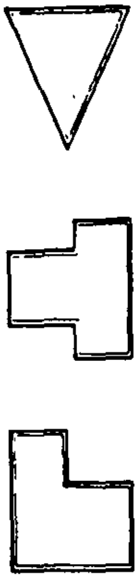


Fig.7. Plantas Irregulares de Edificios

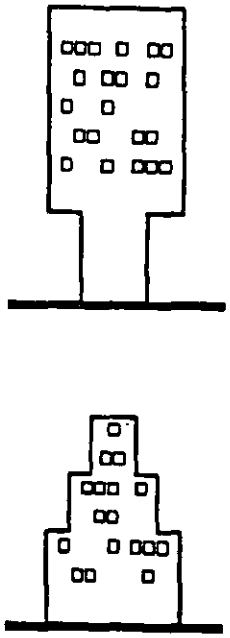
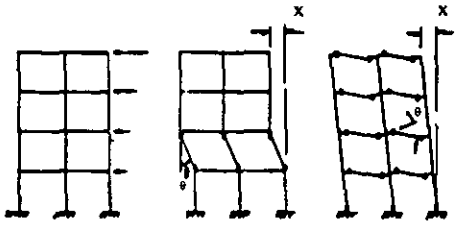


Fig.8. Elevaciones Irregulares de Edificios



a) Marco y cargas laterales

b) Mecanismo de movimiento lateral de columna

c) Mecanismo de movimiento lateral de viga

Si $X_u / X_y = 4$ $\theta_u / \theta_y = 122$ $\theta_u / \theta_y = 18$

Xu = desplazamiento último
Xy = desplazamiento en la primera fluencia
• = articulación plástica

Fig.9. Posibles Mecanismos de un Marco Rigido, Sujeto a Carga Lateral (Ref. 4)

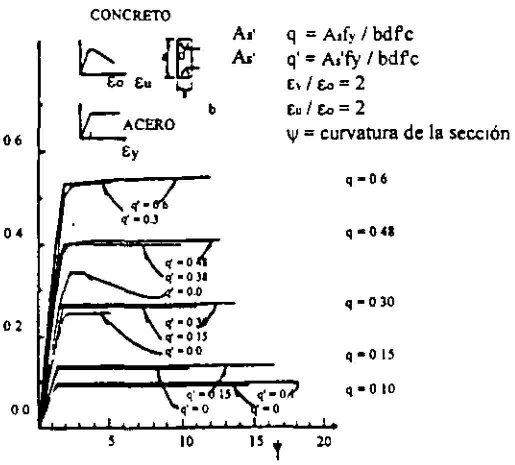
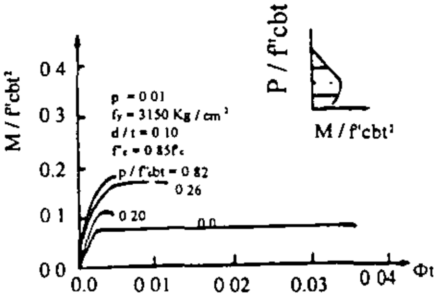
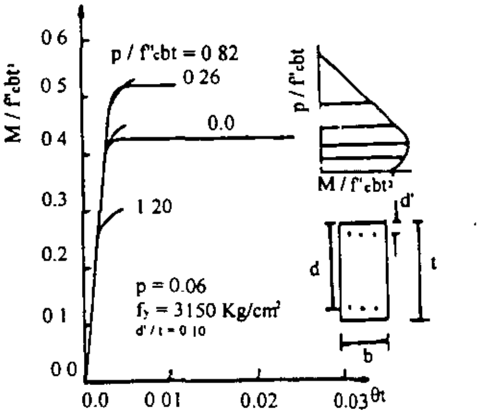


Fig.10. Relaciones Momento- Curvatura para Secciones de Concreto Reforzado, Sujetas a Flexión. (Ref.4)



a) BAJA CUANTIA DE REFUERZO LONGITUDINAL



b) ALTA CUANTIA DE REFUERZO LONGITUDINAL

Fig.11. Relaciones Momento Curvatura de Secciones de Concreto Sujetas a Flexo - Compresion (Ref.4)

FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

XXVI CURSO INTERNACIONAL DE INGENIERIA SÍSMICA

MOUDLO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

DAÑOS POR SISMO EN ELEMENTOS " NO ESTRUCTURALES "

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DAÑOS POR SISMO EN ELEMENTOS "NO ESTRUCTURALES"*

Enrique del Valle Calderón.+

Los muros divisorios y exteriores hechos a base de mampostería y considerados como elementos "no estructurales" por el ingeniero estructurista al diseñar la estructura, pueden modificar considerablemente la respuesta de los edificios al ser sometidos a temblores si son construídos sin holguras adecuadas entre estos elementos y la estructura. Es necesario llevar a cabo estudios experimentales con objeto de establecer el mejor método para eliminar este efecto y la forma en que deben ser soportados estos muros.

INTRODUCCION

Para los fines de este artículo se definirá como elementos no estructurales a todas aquellas partes arquitectónicas de un edificio tales como muros, fachadas, plafones, ductos, tuberías y otros, que no contribuyen "teóricamente" a la resistencia y rigidez necesarias para absorber las cargas a que puede estar sometido, por haberlo decidido así el ingeniero estructurista, ya que al iniciarse el proyecto de la estructura el ingeniero debe definir que partes del edificio van a contribuír a resistir las cargas y que partes no lo harán, y elaborar modelos matemáticos acordes con esas hipótesis; en algunas ocasiones los muros, fachadas y ductos verticales pueden ser considerados parte de la estructura y su contribución a la resistencia y rigidez deben incluírse en el análisis.

En un sentido más amplio algunos autores consideran también como elementos no estructurales a los equipos mecánicos y eléctricos y otros dispositivos fijos a la estructura. Sin embargo, los considerados en este trabajo pueden cambiar considerablemente la respuesta de los edificios durante temblores cuando se construyen violando las hipótesis hechas por el ingeniero con respecto a su contribución a la resistencia y rigidez.

En la arquitectura moderna se ha generalizado el empleo de estructuras -

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esqueléticas que usan columnas como elementos verticales de apoyo, ya que permiten mayor libertad en la distribución de los espacios de cada piso de los edificios, modificando el sistema constructivo tradicional que utiliza algunos muros divisorios y de fachada, contínuos en toda la altura del edificio, para soportar una buena parte de las cargas verticales y horizontales a que se verá sometido éste durante su vida útil; el empleo de muros - conduce en general a estructuras más rígidas, que reducen los movimientos debidos a fuerzas laterales producidas por viento o sismo, en comparación - con los que se presentan en las estructuras esqueléticas.

En los edificios modernos la mayoría de los muros divisorios son fabricados con materiales ligeros (como tabla roca, por ejemplo) pero los muros exteriores y aquellos permanentes alrededor de las zonas de escaleras y elevadores son hechos normalmente de mampostería o de concreto. Cuando se consideran como elementos "no estructurales", usualmente se especifíca que deben construírse de tal modo que no interfieran con el movimiento de la es-- tructura causado por las fuerzas laterales, con objeto de evitarles daños y -- que colaboren a la rigidez del edificio. Frecuentemente la junta entre la estructura y el muro es pequeña por lo que la separación no es eficiente -- para evitar la participación en la rígidez y el daño a esos elementos durante temblores medios o fuertes. Una gran proporción de los daños en temblores recientes ha ocurrido en muros divisorios, fachadas y plafones "no es-- tructurales", con costos elevados de reparación o reposición (Ref. 1-4).

RIGIDEZ LATERAL DE ESTRUCTURAS ESQUELETICAS.

La determinación de la rigidez lateral de estructuras esqueléticas, o estructuras de marcos, es relativamente simple empleando alguno de los distintos programas de computadora disponibles. El modelo matemático puede -- simplificarse a una serie de marcos planos o bien considerar el trabajo de toda la estructura en tres dimensiones.

La rígidez lateral depende principalmente de las propiedades elásticas de los materiales empleados y de las dimensiones de las vigas y columnas -- que constituyen los marcos, tanto para edificios de concreto reforzado como de acero estructural; sin embargo, hay algunas incertidumbres en relación -

con la contribución a la rigidez. Por ejemplo, no es muy clara la contribución efectiva de losas de concreto reforzado coladas monolíticamente con trabes de concreto o ligadas por medio de conectores de cortante a trabes de acero. También hay ocasiones en que es necesario tomar en cuenta los efectos en la rigidez asociados a las dimensiones de la intersección de trabes y columnas en los nudos de los marcos, ya que pueden cometerse errores importantes al suponer que las trabes y columnas tienen propiedades constantes a lo largo de sus ejes, despreciando la modificación de estas propiedades en la intersección; este efecto es importante por ejemplo en marcos de fachada con trabes muy peraltadas, (ref. 5). Otros factores que pueden modificar considerablemente el comportamiento de las estructuras son: la consideración del momento de inercia efectivo de secciones agrietadas a lo largo de las trabes o columnas (zonas de esfuerzos elevados); la relación entre la altura y el ancho efectivo del marco (sin tomar en cuenta voladizos), que puede conducir a fuerzas axiales importantes en las columnas extremas, que al producir deformaciones verticales importantes reducen la rigidez lateral de los marcos; las deformaciones por fuerza cortante en los miembros o en los nudos deben ser tomadas también en cuenta en algunas ocasiones, así como el efecto de fuerzas axiales en la rigidez angular de las columnas.

Hay programas para tomar en cuenta todos o algunos de los efectos anteriores que proporcionan una buena estimación de la rigidez lateral de una estructura sometida a un conjunto de fuerzas laterales específicas.

Cuando las trabes son rígidas en comparación con las columnas la rigidez lateral tiende a ser independiente de las fuerzas aplicadas, pero a medida que disminuye la rigidez relativa de las trabes la rigidez lateral cambia considerablemente para diferentes conjuntos de fuerzas aplicadas. Blume (ref. 6) propone el uso de un índice de rotación nodal para estimar el tipo de marco de que se trata, esto es, un marco bien definido con las columnas flexionadas en doble curvatura y una articulación virtual en cada entrepiso o "un voladizo disfrazado de marco" con las columnas flexionadas en curvatura simple y sin articulación virtual en cada entrepiso; es bastante común que las vigas sean más flexibles que las columnas. Hay algunos sistemas estructurales en los que la flexibilidad del sistema de piso puede ser crítica; por ejemplo, las losas planas aligeradas soportadas únicamente por columnas, formando marcos rígidos "equivalentes" para resistir las cargas verticales y horizontales. Algunos reglamentos no permiten esto y requieren el empleo de muros de cortante para mejorar el comportamiento del sistema; sin embargo, otros, como el de la Ciudad de México, sí lo permiten. Esto conduce a estructuras muy flexibles lateralmente, que rebasan las deformaciones permitidas por los reglamentos o quedan cerca del límite superior.

Las estructuras esqueléticas tienen aplicación límitada debido a las grandes deformaciones laterales que pueden producir las fuerzas sísmicas; sin embargo, se han empleado con éxito en edificios con más de 40 pisos, como la Torre Latino Americana en la Ciudad de México, (ref. 7). Deben cuidarse especialmente las holguras entre la estructura y los elementos " no - estructurales " para evitar que éstos se dañen con temblores medianos y fuertes así como para que se satisfagan las hipótesis hechas por el estructurista con respecto al comportamiento de la estructura.

RIGIDEZ LATERAL DE ESTRUCTURAS CON MARCOS RELLENOS DE MUROS

Se han hecho varios estudios para estimar la modificación en la rigidez lateral de estructuras de marcos en los que algunos tableros se rellenan con muros de mampostería o de concreto. En general se ha observado que la rigidez lateral se incrementa considerablemente, ya que los muros son muy rígidos en su plano y su rigidez puede ser del orden de diez veces la del marco que lo rodea. Hay además un cambio definitivo en la forma en que el marco resiste las fuerzas laterales, disminuyendo sustancialmente los efectos de flexión. Un muro en un tablero puede idealizarse como un puntal diagonal que se opone a la deformación lateral del marco; hay fórmulas que permiten estimar las dimensiones que deben considerarse para este miembro diagonal "equivalente", (ref. 8).

Puede hacerse el análisis por medio de computadora, tomando en cuenta -

estos miembros adicionales que transforman al marco rígido en un marco contraventeado. La figura 1, tomada de la referencia 8, muestra claramente -- los drásticos cambios que ocurren en los momentos flexionantes y fuerzas -- axiales, así como en las rigideces de entrepiso cuando se incluye la colaboración de un muro de mampostería en el análisis de un marco o cuando se desprecia. Obviamente el diseño de los miembros del marco con los momentos y fuerzas obtenidos sin el muro es inadecuado cuando el muro se construye ligado al marco, por lo que puede esperarse algún daño, en los muros o en el marco, cuando la estructura sea sometida a un sismo, como se ilustra en las figuras 2 y 3. El daño puede ocurrir no solo en la mampostería, que es rígida pero débil, sino también en las columnas o vigas del marco que la rodea, debido, usualmente a las fuerzas cortantes adicionales que se desarrollan - en la zona de intersección.

ANALISIS Y CONSTRUCCION

El diseño de los edificios debe hacerse tomando en cuenta diferentes - estados límite que toman en cuenta aspectos de resistencia y de deformación. Generalmente los requisitos de resistencia permiten el empleo de factores - de reducción por ductilidad para calcular las fuerzas sísmicas de diseño, - para tomar en cuenta las excursiones en el intervalo de comportamiento inelástico de algunas zonas de la estructura y la formación de articulaciones plásticas en ella debidos al temblor de diseño. Sin embargo, para el cálculo de los desplazamientos que experimentará la estructura durante dicho temblor, se considera que los desplazamientos producidos por las fuerzas reducidas deben incrementarse en función del factor de reducción por ductilidad empleado, para obtener los desplazamientos elásticos que se obtendrían con las fuerzas elásticas sin reducir, ref. 9, ya que al excursionar en el intervalo inelástico la estructura perderá rigidez, aumentándose los desplaza- - mientos. Usualmente los reglamentos especifican el máximo desplazamiento - que puede tolerarse como una fracción de la altura de entrepiso. Todos los elementos que contribuyen a la rigidez deben incluírse en estos cálculos -- con objeto de distribuír la fuerza total calculada para cada piso del edificio entre todos los elementos resistentes proporcionalmente a sus rigideces.

Por consiguiente, cuando el ingeniero estructurista considera que los muros divisorios y exteriores de mampostería son "no estructurales" y no los toma en cuenta en sus cálculos de resistencia y rigidez su análisis puede estar muy equivocado si el constructor construye esos muros integrados a la estructura haciéndolos también estructurales.

Esto ocurre frecuentemente debido a falta de comunicación adecuada entre el estructurista y el constructor. Los planos para construcción deben incluír detalles muy claros de como deben construírse los elementos no estructurales, ya que de otro modo, el constructor, que no conoce las hipótesis que se hicieron durante la etapa de diseño, puede construír esos elementos inadecuadamente, modificando completamente el comportamiento de la estructura.

Puede también ocurrir que los muros no estructurales se construyan correctamente desligados de la estructura, pero que las holguras que se dejen sean rellenadas con materiales incompresibles o que estos materiales absorban lechada de cemento durante la construcción y se endurezcan o que los acabados del muro, aplanados o recubrimientos con losetas o marmol, pasen sobre la holgura y la nulifiquen.

El estado del arte reflejado en las especificaciones de los reglamentos puede ser también causa del comportamiento inadecuado de estas holguras. Por ejemplo, el reglamento de Construcciones para el Distrito Federal especificaba en su versión de 1966 fuerzas sísmicas reducidas por efectos de comportamiento inelástico de la estructura, con las que se calculaban sus desplazamientos, sin amplificarlos, como se mencionó anteriormente; las holguras entre la estructura y los muros no estructurales se dejaban en función de estos desplazamientos, los que eran tolerables si no excedían 0.002 veces la altura de entrepiso. La versión de 1976 del mismo reglamento, aún en vigor especifica fuerzas sísmicas elásticas que pueden reducirse empleando un factor de reducción por ductilidad que varía entre 1 y 6, dependiendo del tipo de estructura y de los materiales con que se construye ésta así como del cuidado que se tenga al detallarla. En esta versión del Reglamento los desplazamientos totales se deben calcular multiplicando los obtenidos con las fuerzas reducidas por el factor de reducción por ductilidad; estos desplazamientos totales se comparan con dezplazamientos permisibles, iguales a 0.008 veces la altura de entrepiso. Las fuerzas reducidas actuales son aproximadamente las mismas especificadas por la versión de 1966 para el caso de edificios de marcos rígidos; por consiguiente, las holguras que se dejaron antes de 1976 son insuficientes para este tipo de edificio de acuerdo con la versión 1976 y las fallas están esperando el temblor que las producirá.

En los últimos quince años se han construído en la ciudad de México muchos edificios de mediana y gran altura empleando los sistemas estructurales a base de losas planas aligeradas apoyadas sobre columnas, formando marcos - rigidos "equivalentes" para resistir las cargas verticales y laterales a que hizo mención anteriormente, como estructuras muy flexibles.

Durante el temblor del 14 de marzo de 1979 muchos de estos edificios -- sufrieron daños considerables en muros divisorios y de fachada, ref. 4. Algunos de ellos fueron rigidizados para reducir daños en temblores futuros. pero la mayoría fueron solo reparados superficialmente y probablemente serán dañados otra vez en temblores futuros.

Por lo tanto, el hecho de que la estructura se comporte de manera com-- pletamente distinta a lo supuesto por el ingeniero estructurista debido a -- falta de congruencia entre los modelos matemáticos usados en el análisis y - el edificio real, puede ser peligroso.

Hay casos especiales en que esto se agudiza; uno de ellos es el de edificio en esquina con muros de colindancia no estructurales formando un ángulo, en los que los efectos torsionales por sismo pueden dañar a todo el edificio, si estos muros se hacen estructurales durante la construcción. Otro caso frecuente es el que se presenta en edificios escolares que tienen dos - marcos longitudinales, con muros de altura parcial ligados a la estructura - en uno de ellos y sin muros en el otro, donde la rigidez de las columnas de menor longitud efectiva se incrementa por un factor importante (su rigidez - varía aproximadamente con el cubo de la longitud efectiva, por lo que si - - ésta es de un tercio de la longitud considerada en el análisis, el aumento - en rigidez es del orden de 27 veces), lo que causa su falla por cortante, -- fig. 4, y efectos torsionantes en todo el edificio.

Considero que se necesita investigar las mejores maneras de evitar las situaciones antes mencionadas para recomendar detalles prácticos de como -- construír los muros, tomando en cuenta los puntos de vista del estructurista, del arquitecto y del constructor, ya que aunque se han propuesto algunos detalles, refs. 8-9, en ocasiones son costosos y dificiles de construír.

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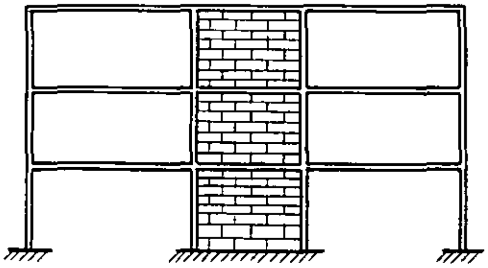
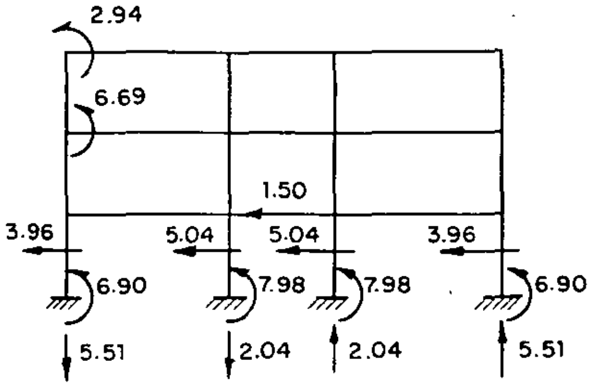
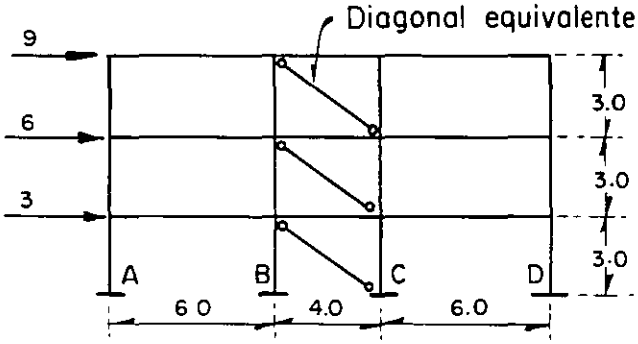


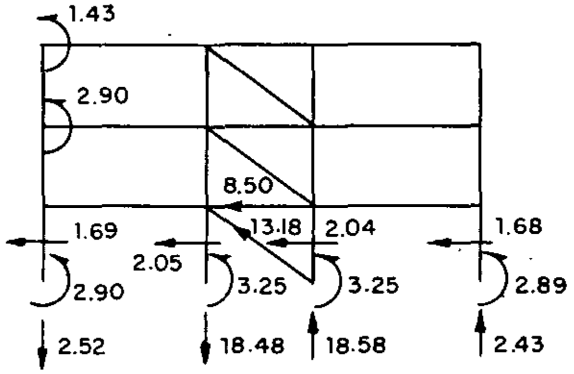
Sección de columnas 0.30 x 0.30 , sección
de vigas 0.25 x 0.50 , f'c = 210 kg / cm²
Muros de mamposteria ,0.15 cm de espesor



a) Sin diagonales



Fuerzas en ton , longitudes en m .

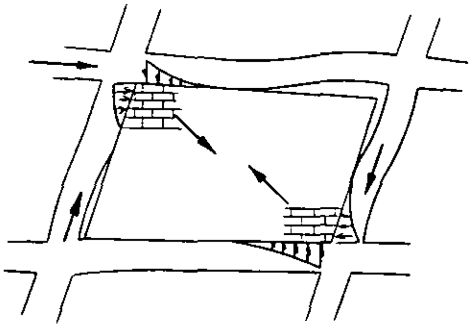


b) Con diagonales

Fuezas en ton ; momentos flexionantes en ton-m

Algunos resultados del análisis del marco

NIVEL	a) SIN DIAGONALES		b) Con DIAGONALES	
	Desp. Lateral cm	Rigidez Ton /cm	Desp. Lateral cm	Rigidez Ton / cm
3	3.72	10.2	1.47	25.4
2	2.84	10.3	1.11	27.2
1	1.39	13.0	0.56	32.1



Muro de mamposteria confinado
por marco

Fig. 1 Marco con muro de mampostería

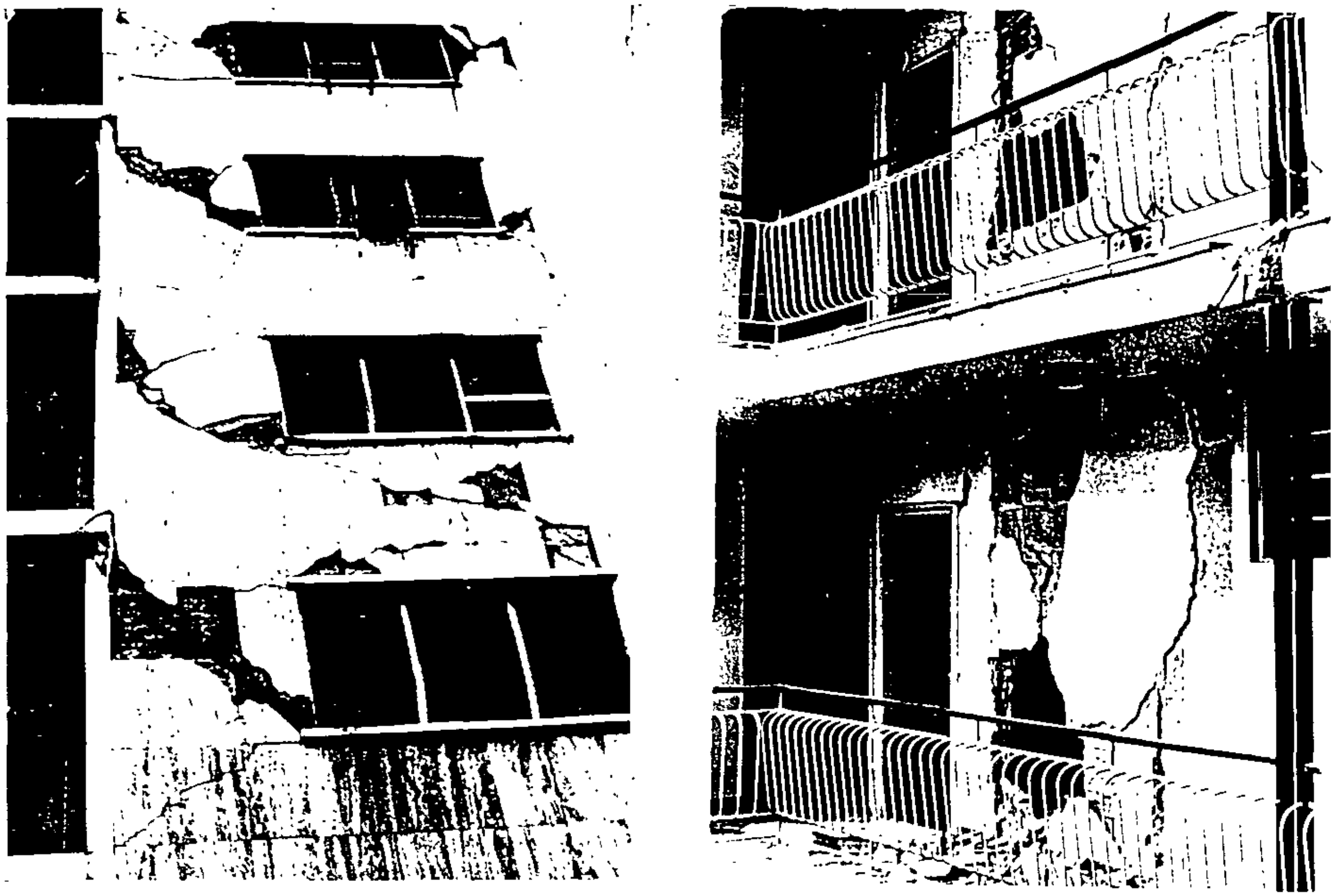
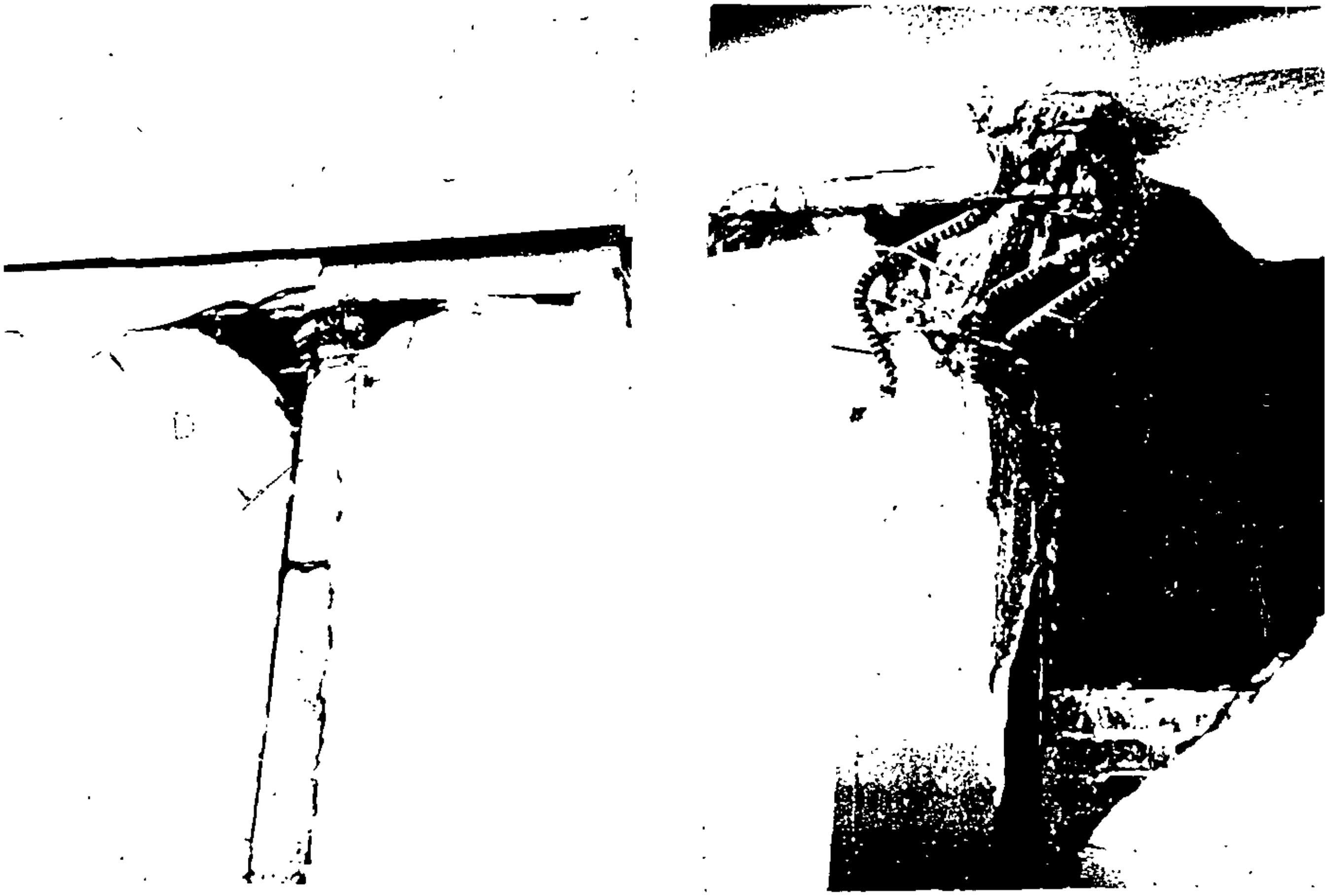


Fig 2 Daños en fachadas "no estructurales"



g 3 Daños a muros no estructurales así como a elementos estructurales (Nótese el ancho de la diagonal "equivalente")

Fig 4 Daños debidos a muros de altura parcial

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CURSOS ABIERTOS

XXVI CURSO INTERNACIONAL DE INGENIERIA SÍSMICA

MOUDLO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

COMPORTAMIENTO Y DISEÑO SISMO - RESISTENTE DE ESTRUCTURAS DE CONCRETO PREFABRICADO Y PRESFORZADO

EXPOSITOR: ING. RENE CARRANZA AUBRY
PALACIO DE MINERIA
SEPTIEMBRE DEL 2000

Mayo 21, 1999. S.M.I.S.

COMPORTAMIENTO Y DISEÑO SISMO-RESISTENTE DE ESTRUCTURAS DE CONCRETO PREFABRICADO Y PRESFORZADO

RESUMEN.-

1.- ESTRUCTURAS PREFABRICADS Y PRESFORZADAS
PROS
CONTRAS

2.- ANALISIS Y DISEÑO.

3.- CONEXIONES
COLS. - CIMENTACION
COLS. - TRABE
TRABES - LOSAS
ANTES Y AHORA

4.- CODIGOS.- PUENTES Y ESTRUCTURAS RETICULARES Y MUROS DE CARGA-CORTANTE.

5.- CONCLUSIONES.-

1.- ENTENDIENDO POR "ESTRUCTURAS PREFABRICADAS", AQUELLAS QUE SE REALIZAN EN UN LUGAR DIFERENTE A SU POSICION FINAL, Y EN UN TIEMPO ANTES DE QUE SE NECESITEN, Y PRESFORZADO AQUEL ELEMENTO AL QUE SE LE INDUCEN ESFUERZOS CONTRARIOS A LOS QUE DEBERA SOPORTAR EN SU CONDICION FINAL DE TRABAJO, PODEMOS DECIR QUE COMO TODA SOLUCION TIENE SUS PROS Y SUS CONTRAS

A PRIMERA VISTA SALTA QUE LOS PRINCIPALES "CONTRAS" SON QUE EL PREFABRICADO DEBERA, ADEMAS DE SU FABRICACION, QUE CASI SIEMPRE REQUIERE, ADEMAS DE UN ANALISIS Y DISEÑO MAS CUIDADOSO, SEA TRANSPORTADO, MONTADO Y CONECTADO PARA ASI INTEGRAR UNA ESTRUCTURA.

LO ANTERIOR AUMENTA EL COSTO EN COMPARACION A LA ESTRUCTURA COLADA IN SITU, YA QUE ESTA NO REQUIERE DE LOS PASOS ANTERIORMENTE DESCRITOS, POR LO TANTO DE NO SER LAS VENTAJAS QUE PRESENTA, APARENTEMENTE NO TENDRIA SENTIDO PREFABRICAR ESTRUCTURAS DE CONCRETO PRESFORZADO.

LAS VENTAJAS PRINCIPALES DE LA PREFABRICACION Y EL PRESFUERZO, SON SIN DUDA LAS SIGUIENTES:

1.- MEJOR CALIDAD.- EN LAS PLANTAS SE CUENTA CON LABORATORIOS PARA CONTROLAR LA CALIDAD DE LOS MATERIALES INTEGRANTES DEL CONCRETO. PARA AL FIN DETERMINAR LA f'c DEL MISMO. EL ACERO DE PRESFUERZO, ADEMAS DEL CERTIFICADO DE CALIDAD DEL FABRICANTE, AL TENSARLO COMPROBAMOS SU RESISTENCIA Y SU MODULA DE ELASTICIDAD. AL PRESFORZAR, UTILIZAMOS CONCRETO CON UNA f'c MINIMA DE 300 KG./CM2, LLEGANDO ACTUALMENTE HASTA 450 KG./CM2 SIN GRANDES PROBLEMAS.

2.- MENOR PESO Y MAYOR EFICIENCIA.- CON SECCIONES ADECUADAS PARA MOMENTOS FLEXIONANTES Y CORTANTES ELEVADOS. LOS ELEMENTOS PREFABRICADOS Y PRESFORZADOS EN TRABES Y LOSAS. PUEDEN PROYECTARSE CON RELACION DE CLARO A PERALTE (L/P), DE 1/20 , O MAYORES, COSA QUE NO SE PUEDE HACER CON CONCRETO REFORZADO COLADO IN SITU.

3.- EL USO REPETIDO DE LOS MISMOS MOLDES. HACE MUY ECONOMICO EL CIMBRADO DE LAS PIEZAS, EN COMPARACION A LA CIMBRA Y OBRA FALSA REQUERIDA EN LAS OBRAS.

4.- LA SUPERVISION DEL TRABAJO REALIZADO EN PLANTA A NIVEL DE PISO. ES MEJOR QUE LA QUE SE REALIZA EN OBRAS, A MEDIDA DE QUE SEAN MAS ALTAS.

5.- "TIEMPO".- DE ACUERDO A EXPERIENCIAS MUY COMPROBADAS, UNA OBRA PREFABRICADA, PUENTE O ESTRUCTURA RETICULAR O CON MUROS DE CARGA Y CORTANTE, SE REALIZA CUANDO ESTA BIEN PLANEADA, EN UNOS CUANTOS DIAS, EN COMPARACION AL NECESARIO CUANDO LA MISMA OBRA SE CUELA IN SITU.

"TIEMPO ES DINERO".- CUANDO SE RECONOZCA QUE UNA OBRA REALIZADA EN MENOR TIEMPO, SU COSTO FINANCIERO Y SU PUESTA EN MARCHA Y PRODUCCION, AMPLIAMENTE COMPENSA EL GASTO EXTRA EN TRANSPORTE, MONTAJE Y CONEXION.

6.- SEGURIDAD.- CON LOS AVANCES ACTUALES. EL PROBLEMA DE CONEXION SE HA RESUELTO PARA QUE TODA OBRA PREFABRICADA, DEBIDAMENTE CONECTADA, TENGA UN COMPORTAMIENTO EXCELENTE ANTE CARGAS GRAVITACIONALES Y HORIZONTALES TALES COMO SISMOS Y VIENTOS.

7.- PRODUCCION INDUSTRIALIZADA.- A MEDIDA QUE SE REQUIERAN GRAN CANTIDAD DE PIEZAS DE MOLDES EXISTENTES Y FACILMENTE ADAPTABLES, LOS PROCESOS INDUSTRIALIZADOS MEJORAN LOS TIEMPOS Y COSTOS, ASI COMO LA CALIDAD DE LOS PRODUCTOS.

CONEXIONES

CIMENTACION - COLUMNA.

a) CIMENTACION COLADA IN SITU
- PERNOS AHOGADOS
- VAINAS Y EPOXICOS
- BARBAS EN COLUMNA
- PLACAS SOLDADAS

b) CIMENTACION MIXTA.- CANDELERO PREFABRICADO CON ZAPATA IN SITU.

c) CIMENTACION PREFABRICADA.- ZAPATA - CANDELERO PREFABRICADO.

COLUMNA - TRABES.-

a) MENSULAS COLADAS MONOLITICAS CON FUSTE COLUMNA EN 1 - 2 - 3 Y 4 DIRECCIONES.

b) REQUIEREN PLACAS SOLDADAS A VARILLAS Y SOLDADURA DE PLACA DE MENSULA A PLACA DE TRABE.

c) POSTENSADAS.

d) PERNOS Y CONECTORES TIPO SPLICE.

e) SOLDADURA DE VARILLAS ENTRE SI.

f) CONEXIONES FUERA DE ZONA DE NUDOS.

g) CONEXION SEPSA.

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CONEXIÓN TRABE-COLUMNA DE ELEMENTOS PREFABRICADOS SIN PERNOS NI SOLDADURA

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RESUMEN

Éste trabajo se centra en las conexiones entre elementos prefabricados que es uno de los aspectos más importantes para el buen funcionamiento de este tipo de estructuras. Se propone un tipo de conexión innovadora que está exenta de placas de acero y soldadura, lo que nos permite evitar los cambios bruscos de secciones y materiales, además de que es más agradable a la vista al simular a una estructura monolítica. Se presenta la aplicación de la conexión propuesta en dos obras diferentes, en donde se obtuvieron excelentes resultados en cuanto a los tiempos de ejecución, comportamiento y economia.

SUMMARY

This work meant about to the connection between precast concrete elements that is one of the most important aspects in the development of precast concrete structures. A new connection that does not need to be welded is proposed without changes on precast elements sections neither materials. This connection avoids fragile failures. Also its appearance as a monolithic structure connection seems pleasant. The propose connection is showed in two different buildings where excellent results were obtained in terms of time, performance and economy

INTRODUCCIÓN

Él ingeniero involucrado en el análisis, diseño, prefabricación, transporte y montaje de una estructura reticular prefabricada de concreto tiene como un gran reto el conectar las piezas prefabricadas que la integran, especialmente la unión columna-trabe, ya que en algunas ocasiones concurren hasta cuatro trabes en diferente dirección en una misma columna en cada nivel de la estructura.

Uno de los mayores problemas que se presentan en una estructura prefabricada, no es un análisis deficiente de la estructura, sino la poca importancia que se le ha dado al diseño de las conexiones entre los diferentes elementos que la forman. Debemos tomar en cuenta que la conexión recibirá las descargas de cada elemento que se conecte a ella y deberá ser capaz de soportalas y transferirlas a los demás elementos (Arthur H. Nilson, 1982)

En los pncipios de la prefabricación en México, únicamente se prefabricaban sistemas de piso como son vigueta y bovedilla, losa doble te y losas extruidas, los cuales se colocan simplemente apoyados sobre trabes de soporte por lo que no se tenia la necesidad de diseñar sistemas de conexión entre elementos. Sin embargo, con el paso del tiempo se fueron demostrando las ventajas de las estructuras prefabricadas sobre las tradicionales. Por ejemplo: Un menor tiempo de ejecución de la obra y por lo tanto una recuperación más rápida de la inversion, claros más grandes con elementos esbeltos y obras más limpias durante su ejecución. Estas ventajas han hecho que, hoy en día, las estructuras sean casi totalmente prefabricadas y sea de suma importancia las conexiones entre los diferentes elementos que las forman.

CONEXIONES PREFABRICADAS

Características Generales

El diseño correcto de una conexión es indispensable para que la estructura trabaje de acuerdo al modelo físico y matemático con el cual se realizó el análisis estructural A través de los años se ha resuelto este problema de conexión usando el ingenio y llegando a diferentes soluciones, tales como, ménsulas de concreto o acero las cuales pueden ser visibles por debajo de la trabe o se pueden disimular con éstas; las trabes pueden estar simplemente apoyadas o se les puede dar continuidad por medio de pernos, soldadura o postensado (figura 1).

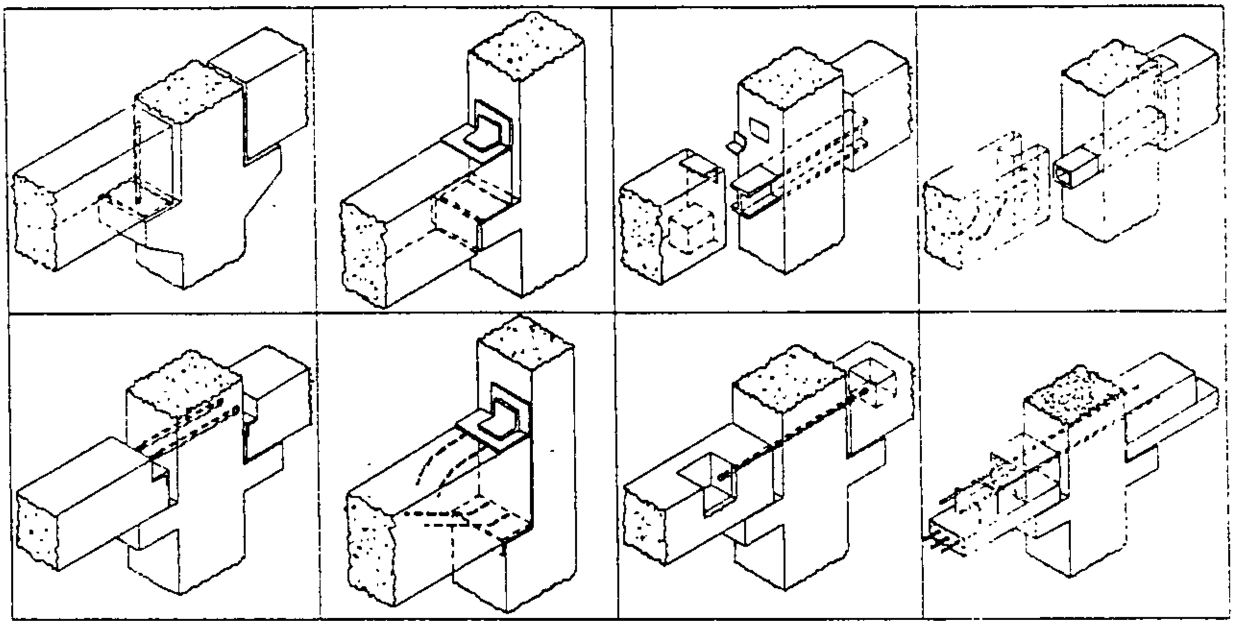

Fig. 1 .- Diferentes tipos de conexión columna-trabe empleadas.

Pocas conexiones han resuelto el problema de una forma totalmente satisfactoria, ya que en cualquier tipo de conexión se busca simplificar las diferentes etapas de una obra, como son:

Fabricación:

-Que sea sencilla, es decir, que la fabricación de los diferentes elementos que la forman, como son trabes y columnas, no se complique con accesorios soldados a su acero principal como placas de acero, elementos de anclaje para postensado, etc.

-Que no aumente la tipificación de columnas, es decir, que las columnas no sean muy diferentes geométricamente entre sí, ya que esto representa un mayor trabajo de gabinete además de que complica la coordinación entre la fabricación y el montaje de las piezas.

-Que no requiera del uso de muchos planos en obra para poder realizarse, ya que esto, además de requerir un mayor trabajo de gabinete es poco práctico de realizar en la planta y posteriormente en obra.

-La fabricación de los moldes debe ser sencilla para evitar retrasos en su elaboración y que no sean muy costosos. Además no se debe requerir una gran cantidad de ellos ni deben requerir muchos cambios entre cada colado.

Transporte:

-Debe poder realizarse de una forma eficiente, es decir, que se puedan transportar el mayor número de piezas por viaje, ya que esto reducirá el número total de fletes que se tienen que realizar y por consecuencia bajará el costo por este concepto.

-No se deben requerir de accesorios especiales para sujetar las piezas a las plataformas, ya que éstos son costosos y dificultan la carga y descarga de las piezas.

Montaje:

-La maniobra de montaje de las diferentes piezas se debe poder realizar en una forma rapida y sencilla, ésto evitará tiempos muertos de equipo, maquinaria y personal, lo que nos permitirá un sustancial ahorro en el tiempo de ejecución de la obra.

-Se debe evitar el uso de equipo y mano de obra especializada como soldadores y personal de postensado, ya que esto eleva el costo de la obra y puede afectar el tiempo de ejecucion en caso de no tener suficiente personal capacitado

-Evitar o minimizar el uso de soldadura de campo, ya que, requiere de un estricto control de calidad, se lleva más tiempo en su ejecución y además su costo es muy elevado

-Que requiera poco o nulo soporte temporal de los elementos, ya que esto retrasa los tiempos de ejecución.

-La apariencia final de la conexión debe ser agradable a la vista, es decir, debemos tratar de ocultar los elementos de sujeción, como son placas de acero, soldaduras, etc

Principales tipos

A través de los años las conexiones en las estructuras prefabricadas se han resuelto, en forma global , en cuatro grandes grupos (IMCYC, 1966,1976,1981; PCI, 1988).

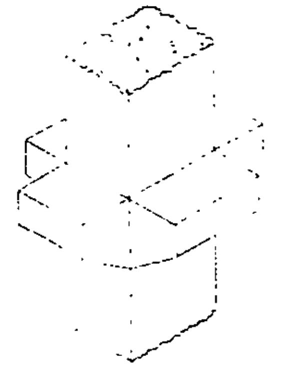

FIG No [illegible] COLUMNA PREFABRICADA CON CUATRO MENSULAS CORTAS

Con ménsula corta.-Ésta es una conexión cercana al paño de la columna. Uno de los principles problemas constructivos que se presentan es al momento de colar las columnas, cuando se requiere ménsulas en las cuatro direcciones, ya que debido a la gran concentración de acero en esa zona se require de soldadura tanto dentro de las piezas a conectar como la unión entre ellas (soldadura de campo), lo que hace que sea una conexión poco dúctil (Cuevas y Robles, 1986) Como normalmente en una obra las columnas tienen diferente número y posición de las ménsulas, es necesario cortar los costados y el fondo de los moldes y taparlos cuando no se requieran. Lo anterior provoca que los tiempos de colado se retrasen y obliga a tener un mayor número de moldes aumentando el costo de la obra El transporte se vuelve menos eficiente para las columnas debido a que las ménsulas aumentan el ancho de las piezas, por lo que el número de piezas que por geometria se pueden transportar por viaje es menor, aumentando el precio por este concepto En la maniobra de montaje también se tienen algunos problemas adicionales cuando no se tienen ménsulas en forma asimétrica, ya que al momento de izar las columnas para insertarlas en su posicion en la cimentacion estas se inclinan un poco por el desequilibrio de peso fuera de su eje longitudinal (figura 2).

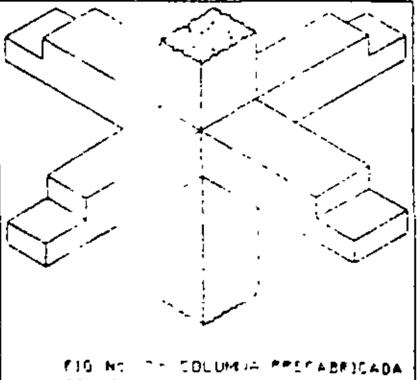

FIG No [illegible] COLUMNA PREFABRICADA CON CUATRO MENSULAS LARGAS

Con ménsula larga.-Ésta es una conexión alejada del paño de la columna que busca llevar la unión a una zona donde el momento es menor. Presenta los mismos problemas que la anterior (ménsula corta) aunque algunos en forma más severa. Cuando se tienen ménsulas largas en las cuatro direcciones se presenta un grave problema al momento de realizar el colado de la columna, ya que las ménsulas que quedan hacia el fondo y hacia arriba son dificiles de realizar, los problemas de transporte y montaje son también mayores, debido a que la geometria de las columnas las hace poco manejables. Todo lo anterior hace que los precios de fabricación, transporte y montaje de las piezas se incrementen aún más (figura 3).

Con postensado.-Este tipo de conexiones no tiene problemas de ductilidad, las columnas pueden o no estar provistas de ménsulas. Cuando tienen ménsulas presentan los problemas que anteriormente mencionamos, sin embargo cuando las columnas no cuentan con ménsulas de apoyo, las trabes tendrán que soportarse temporalmente por medio de apuntalamiento. Se debe tener mucho cuidado con la posición de los ductos y anclajes para el postensado durante el diseño y fabricación de las trabes y columnas, ya que éstos deben coincidir perfectamente al momento de montar las piezas para que permitan el paso de los cables y se realice el postensado sin causar momentos adicionales a la estructura Todo lo anterior aunado al alto costo del postensado incrementa el tiempo y costo final de la obra (figura 4).

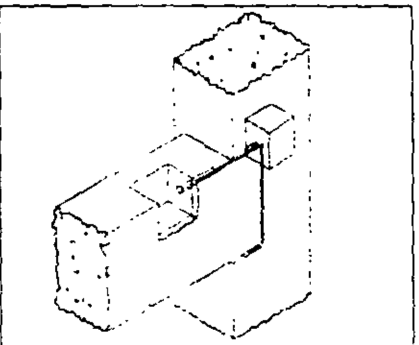
FIG No 4 - CONEXION TRABE-COLUMNA POSTENSADA

Solución propuesta.-Este tipo de conexión proporciona un nodo que es el más parecido a los nodos monolíticos de las estructuras tradicionales. La fabricación de las columnas se simplifica en gran medida, ya que al no contar con ménsulas no es necesario cortar el fondo ni los costados de los moldes. Ésto nos permite disminuir el número total de moldes necesarios al poder darles un mayor número de usos sin grandes cambios No se requiere dejar ahogados en columnas ni trabes anclajes especiales ni elementos soldados al acero de refuerzo El transporte se optimiza al llevar una mayor cantidad de piezas por cada viaje, bajando los costos por este concepto. El montaje de las columnas se facilita al coincidir su eje longitudinal con el centro de gravedad, lo que evita inclinaciones de las piezas al momento de izarlas No es necesario el empleo de personal especializado para el armado y colado de los nodos, pudiéndose utilizar el mismo personal que se emplea en las construcciones tradicionales (figuras 5 y 6) Todo lo anterior permite obtener una mayor economía de tiempo y costo además de una mejor apariencia final a un bajo costo con respecto a los otros tipos de conexiones (fotos 5 y 6).

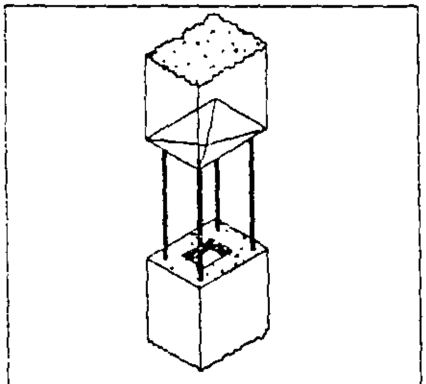
FIG No 5 - COLUMNA PREFABRICADA CON "VENTANA"

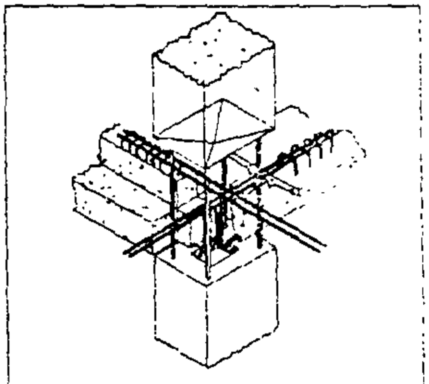
FIG No 6 - CONEXION TRABE-COLUMNA PROPUESTA

Existen una infinidad de tipos de conexiones, pero las anteriores son las más comúnmente usadas en el medio. En la Tabla 1 podemos apreciar una comparativa de las principales características de las conexiones Sin embargo, la necesidad de encontrar una conexión más eficiente sobretodo ante los efectos sísmicos nos llevó a desarrollar una conexión diferente, que nos permite tomar la eficiencia de una estructura prefabricada sin perder las bondades estructurales de una colada "in situ". No debemos olvidar que el Reglamento de Construcciones del Distrito Federal considera que las estructuras prefabricadas deben tener desplazamientos menores que las estructuras coladas "in situ", y ésto se debe en parte a las conexiones. Todos los tipos de conexiones que hasta ahora se han estado usando son a base de pernos o soldadura y presentan un tipo de falla frágil, que es algo que se quiere evitar.

Tabla 1 .- Comparación de grupos de conexiones.

TIPO DE CONEXIÓN	FABRICACIÓN	TRANSPORTE	MONTAJE	REQUERIMIENTOS	APARIENCIA
MÉNSULA CORTA	Complicada, sobre todo cuando se tienen ménsulas en las cuatro direcciones	Poco eficiente, se reduce el número de columnas por viaje	Se complica un poco por inclinaciones al momento de izar las columnas	Un mayor número de moldes, cortar en diferentes partes los moldes. Requiere soldadura para armado de trabes y columnas, también soldadura de campo	Ménsulas visibles en la parte inferior de las trabes, placas de acero y soldadura visibles
MÉNSULA LARGA	Muy complicada, sobretodo cuando se tienen ménsulas en las cuatro direcciones	Complicado se minimiza el número de columnas por viaje y se necesitan accesorios especiales para su transporte	Complicado debido a la geometría de las columnas son poco manejables	Un mayor número de moldes y cortarlos en diferentes partes. Requiere soldadura en armados y de campo	Junta visible en unión con trabes, soldadura y placas visibles
POSTENSADO	Se complica al requerirse gran precisión para fijar los ductos en el armado de trabes y columnas	En caso de que las columnas no cuenten con ménsulas se simplifica	En las columnas se simplifica cuando no lleven ménsulas, pero se requiere gran precisión para hacer coincidir los ductos de trabes y columnas, lo cual retrasa los tiempos de ejecución	Personal especializado para hacer los trabajos de postensado. En caso de contar con ménsulas se requiere apuntalamiento temporal	Juntas visibles en unión de columnas y trabes
CONEXIÓN PROPUESTA	Se simplifica, se puede dar un mayor número de usos a los moldes al no tener que cortarlos, por lo que se requiere un menor número. No es necesario fijar ningún tipo de accesorio al armado de las trabes y columnas	Se optimiza al poder transportar una mayor número de columnas por viaje	Se simplifica mucho. No se presentan problemas de inclinación al izar las columnas, se optimiza los tiempos de ejecución	NO se necesita personal especializado ni soldadura, se pueden emplear fierreros y albañiles que estén trabajando en la cimentación	Como si fuera una conexión monolítica NO quedan visibles ningún tipo de junta

X

DESCRIPCIÓN DE CONEXIÓN PROPUESTA

En una estructura prefabricada es deseable la continuidad de los elementos y que no tengan cambios bruscos de secciones y materiales, razón por la cual desarrollamos una conexión que se asemeja lo mejor posible al colado "in situ". La conexión propuesta es una variante en el sistema de construcción y fabricación de los elementos prefabricados.

Las preparaciones de los elementos se hacen en la planta de producción, las trabes están provistas del acero positivo necesario de acuerdo al diseño y pueden contar, si se desea, con el acero negativo. inmediatamente después del montaje de las trabes una cuadrilla de fierreros comienza con el armado, cimbrado y colado de la conexión en obra, mientras se continúa con las maniobras de montaje

Se deben tomar en cuenta ciertas consideraciones estructurales al utilizar este tipo de conexiones, ya que las primeras condiciones de apoyo y empotramiento de los elementos no son iguales a las condiciones finales Las columnas tienen una parte hueca donde su rigidez, antes de colarse la conexión, es considerablemente menor que el resto de su sección; las trabes deben diseñarse para dos condiciones de apoyo, la primera simplemente apoyada y la segunda con continuidad en la que formarán marcos ortogonales con las columnas.

Si se tiene una estructura de varios niveles se debe revisar la primera etapa de montaje, cuando las columnas estan empotradas en la cimentación y las trabes que están montadas se encuentran simplemente apoyadas, ante cargas accidentales tales como sismo, viento e incluso posibles golpes de otras piezas al momento de montarlas, ya que en ésta etapa las columnas trabajan en cantiliver y puede ser muy peligroso para la estabilidad de la estructura si se presentan desplazamientos grandes También se recomienda en éstos casos, en la medida de lo posible, colar los nodos de los niveles inferiores antes de continuar con el montaje de los niveles superiores

Fabricación

Una de las principales ventajas, es la sencillez de los moldes para fabricar las columnas y el poco trabajo que requieren para su habilitado entre cada colado, lo que hace que se optimicen sus usos con un considerable ahorro. Por otro lado el no necesitar fijar ningún tipo de acccesorio adicional, como son placas de acero o elementos para anclaje de postensado en los armados de trabes y columnas, agiliza la producción.

El colado de las columnas se realiza dejando zonas sin colar (huecas) en los diferentes niveles donde se conectarán las trabes, las que en lo futuro denominaremos "ventanas", en las cuales únicamente continuara el acero principal y si es necesario se colocarán varillas en forma de contraventeo en cada cara. El extremo inferior de las ventanas de las columnas, donde apoyarán las trabes, debe tener una superficie lisa y nivelada para evitar concentracion de esfuerzos al apoyar las trabes correspondientes, y se debe dejar un pequeño hueco rectangular en el centro de la columna que haga las veces de dentellón. El otro extremo de las ventanas debe terminar en forma de punta (piramide invertida)con una inclinación de 30° para facilitar el colado del nodo y evitar que no queden burbujas de aire atrapado, además de no presentar una superficie horizontal de contacto entre concretos de diferentes edades y evitar problemas de cortante.

También se recomienda dejar ahogado un ducto que vaya desde la punta hasta un costado superior de la columna, para permitir la salida de aire cuando se efectue el colado del nodo correspondiente El tamaño de la ventana esta en función del peralte de las trabes que se insertarán, tomándose dos veces el mayor de los peraltes medidos desde la superficie lisa inferior hasta la punta de la parte superior.

La fabricación de las trabes portantes y de rigidez también se facilita ya que tampoco requieren ningun tipo de accesorio especial, lo que agiliza el habilitado de los armados dentro de los moldes En caso de que el ancho de las trabes lo requiera se les fabricará en sus extremos una "nariz" (reduccion de su seccion), en este punto debemos tener especial cuidado ya que el éxito de la conexión depende en gran parte del espacio con el que se cuente para montarlas y conectarlas. En ambos extremos de las trabes, o en la nariz si es el caso, se dejaran ductos transversales de 1/2" de diámetro a una separación de 15 cms. entre cada uno aproximadamente, éstos ductos tienen la finalidad de que una vez montadas las trabes nos permitan colocar unas varillas en forma de ganchos, que abrazaran al acero principal de la columna y funcionarán como estribos de ésta, se recomienda dejar dos lineas de ductos para asegurar que una de las dos quede fuera del acero principal previniendo posibles diferencias de longitudes al montar las columnas. No se recomienda dejar más de dos lineas de ductos para no debilitar la trabes por cortante

Transporte

La principal ventaja de la conexión propuesta en ésta etapa se tiene en las columnas El transporte de las columnas se optimiza de una manera muy eficiente, ya que, al no contar con ninguna mensula se pueden colocar juntas sobre la plataforma, además de que no se necesita fabricar ningún tipo de accesorio especial para fijarlas sobre el tractocamión, de ésta manera se pueden transportar una mayor cantidad de estas piezas por cada viaje y se reduce notoriamente el número total de fletes que se tienen que realizar para transportarlas, con el consecuente ahorro de tiempo y costo.

Montaje

La maniobra de montaje de las diferentes piezas que forman la estructura es bastante ágil al emplear esta conexion Al no tener que emplear una gran cantidad de planos explicativos para colocar las diferentes piezas prefabricadas, se simplifican los trabajos tanto en la elaboración del proyecto como en el campo. No es necesario el empleo de equipo y personal especializado, como sucede en las conexiones postensadas, tampoco se requiere el uso de apuntalamientos o soportes temporales de las trabes y/o columnas y, lo que es más importante, no necesitamos utilizar soldadura de campo que además de presentar un tipo de falla frágil requiere de una estricta supervición, es costosa y necesita de más tiempo para realizarse, ya que normalmente se tienen que soldar las trabes a las columnas antes de poder montar las losas o sistemas de piso, lo que hace que el tiempo total de la maniobra de montaje aumente.

Primeramente se insertan las columnas en los candeleros (huecos) previamente dejados en la cimientación y se fijan temporalmente con cuñas de madera, posteriormente se empotran por medio del colado de la junta entre columna y candelero con un mortero con estabilizador de volúmen, una vez que el colado de empotramiento ha alcanzado la resistencia de diseño se procede a cortar las varillas de contraventeo de las "ventanas", si es que existieran. Ahora las columnas se encuentran listas para recibir a las trabes portantes y de rigidez, estas se insertan inclinandolas lo que sea necesario hasta colocarlas en su lugar, ésta maniobra puede parecer complicada de realizar sin embargo no lo es y en la práctica es bastante rápida de realizar (fotos 1, 2 y 3).

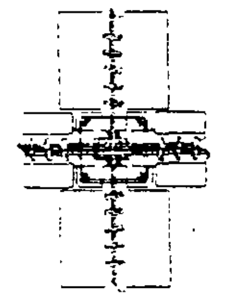

FIG. 7 - PLANTA DE NODO PROPUESTO

Una vez colocadas en su lugar las columnas y las trabes se procede al armado de los nodos correspondientes (foto 4), ésta actividad se traslapa con la maniobra de montaje de los demás elementos de las estructura para evitar tiempos muertos, el armado de los nodos consiste en lo siguiente: Conectar el acero positivo de las trabes por medio de estribos interiores, habilitar el acero negativo correspondiente de cada trabe, ésto representa una ventaja ya que este acero es continuo y se sujeta por medio de estribos abierto previamente ahogados en las trabes, finalmente se colocan los ganchos que pasando através de los ductos dejados en las trabes abrazan al acero principal de las columnas, haciendo la función de estribos (fotos 3 y 4). Una vez terminado el armado de los nodos se procede a su cimbrado y colado, si la geometría de las trabes lo permite se pueden fabricar cimbras metálicas para agilizar el proceso y bajar costos, para el colado se utiliza concreto de la misma resistencia que las columnas con estabilizador de volumen (figuras 7 y 8).

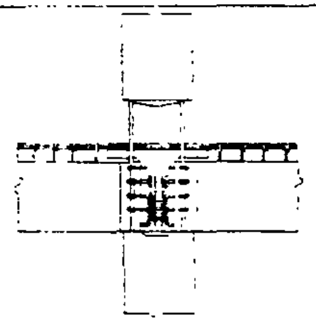

FIG. 8 - ALZADO DE NODO PROPUESTO

El montaje de las losas prefabricadas o el sistema de piso que se este empleando se puede realizar antes o depués del colado de los nodos, dependiendo del diseño de las trabes, únicamente se recomienda colar los nodos de los niveles inferiores antes de montar las losas de los niveles superiores por seguridad de la estructura.

CONCLUSIONES

La utilizacion de la conexión columna-trabe para elementos prefabricados propuesta en este trabajo, tiene muchas ventajas sobre las utilizadas hasta ahora en el mercado, ventajas en la fabricación, transporte, montaje, tiempo, costo y seguridad estructural, que resulta en una herramienta muy poderosa para el desarrollo y repunte de las estructuras prefabricadas en el futuro

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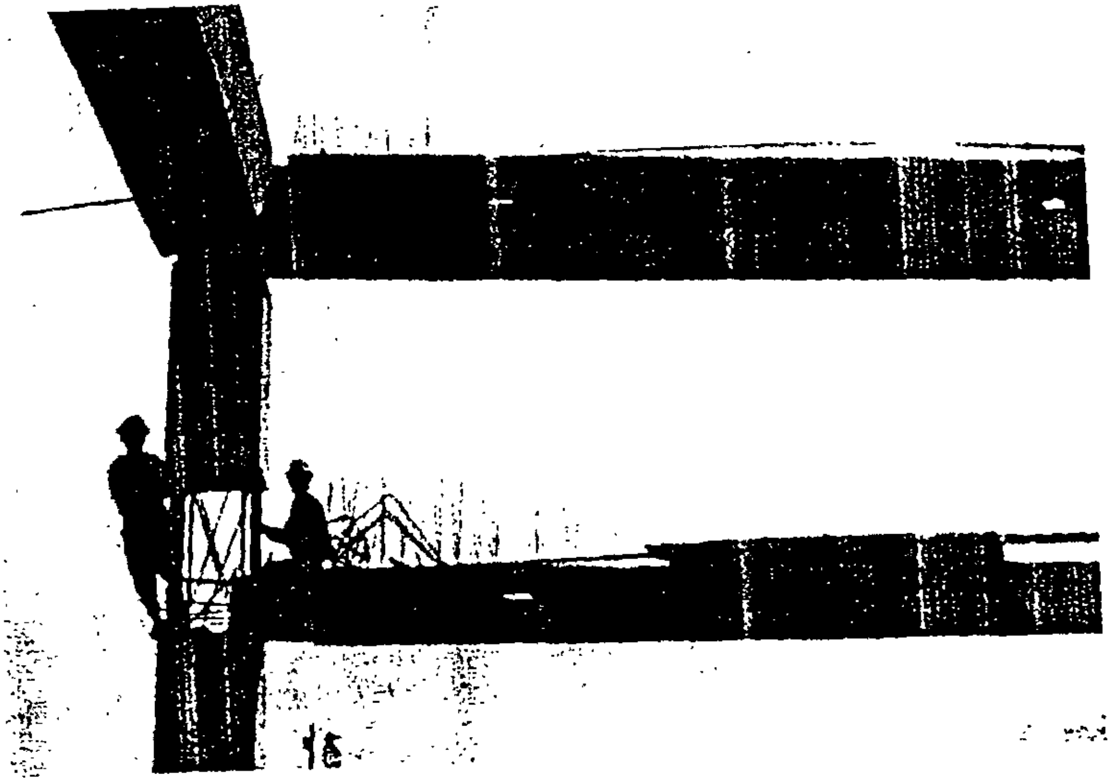

FOTO No. 1.- INSERCION DE TRABES EN "VENTANAS" DE COLUMNAS PREFABRICADAS (CONEXION PROPUESTA)

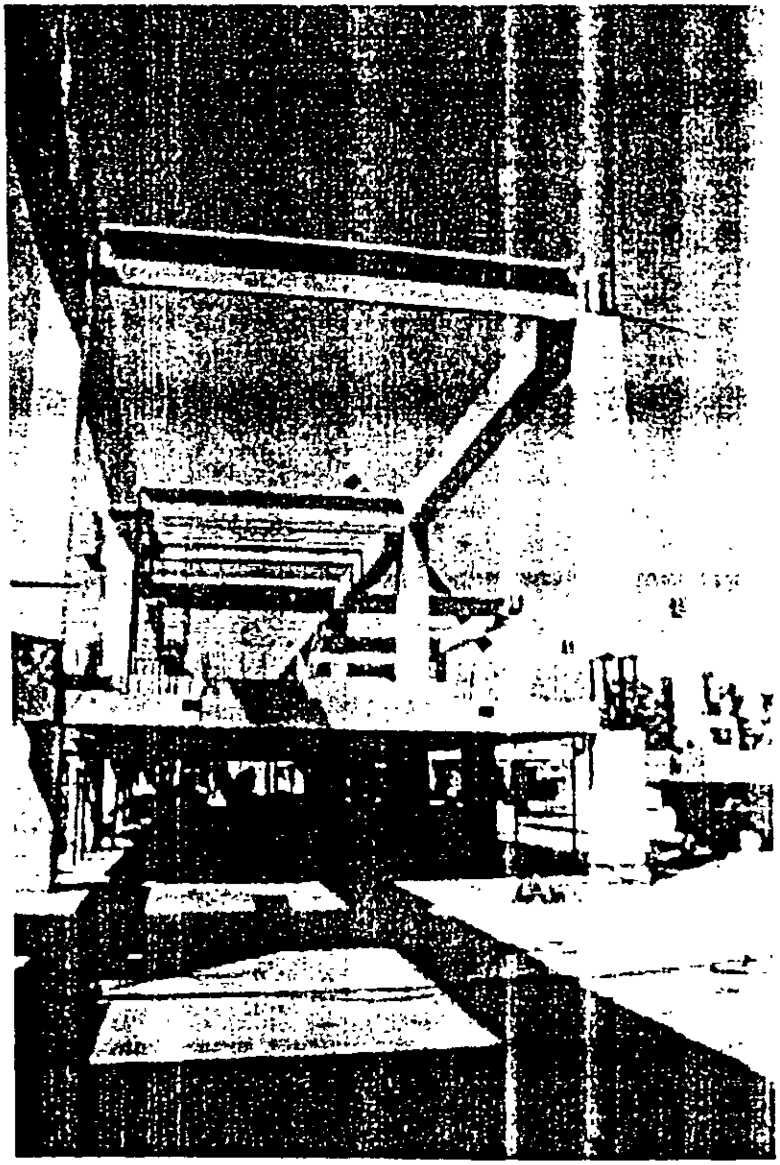

FOTO No. 2.- VISTA DE TRABES INSERTADAS EN "VENTANAS" DE COLUMNAS (CONEXION PROPUESTA)

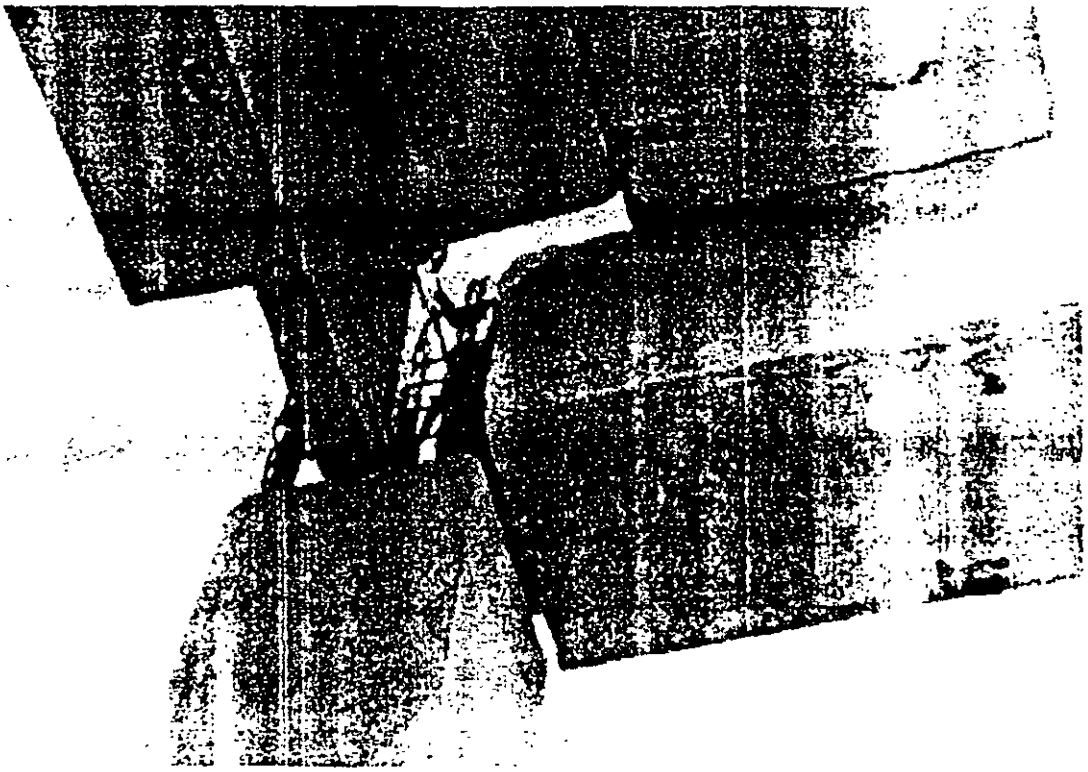

FOTO No. 3.- DETALLE DE MONTAJE DE TRABES (CONEXION PROPUESTA)

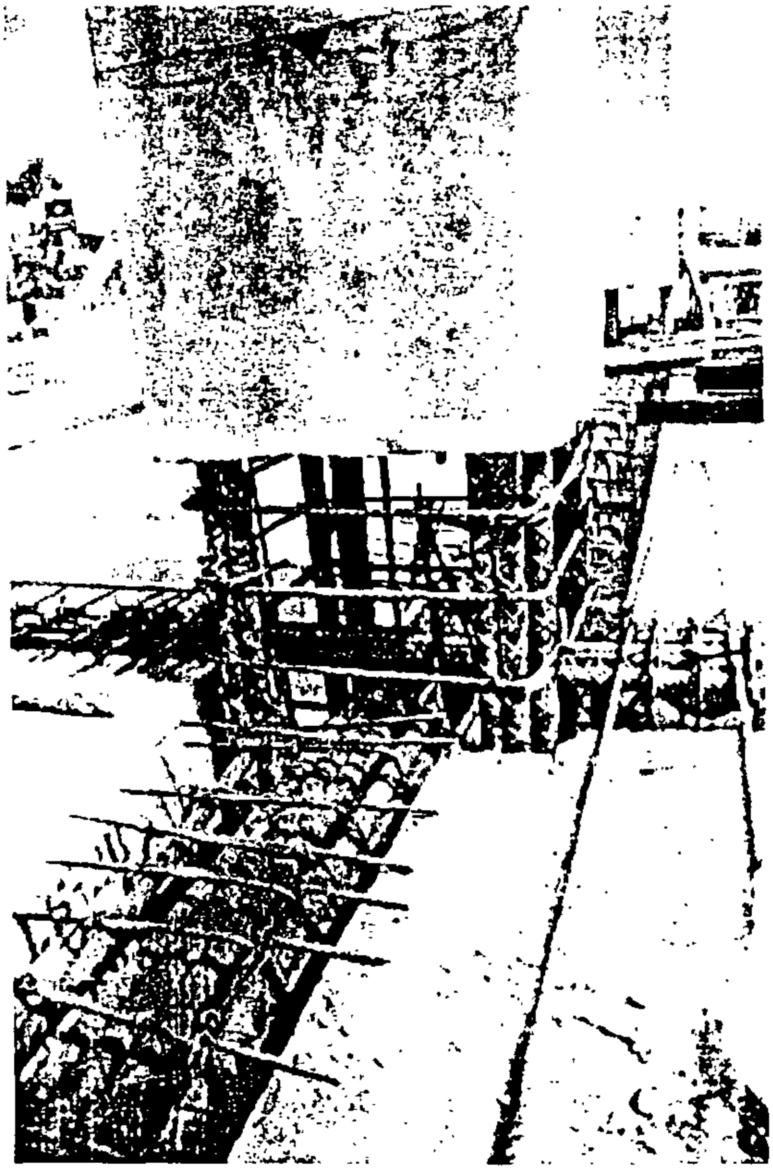

FOTO No. 4.- DETALLE DE ARMADO DE NODO

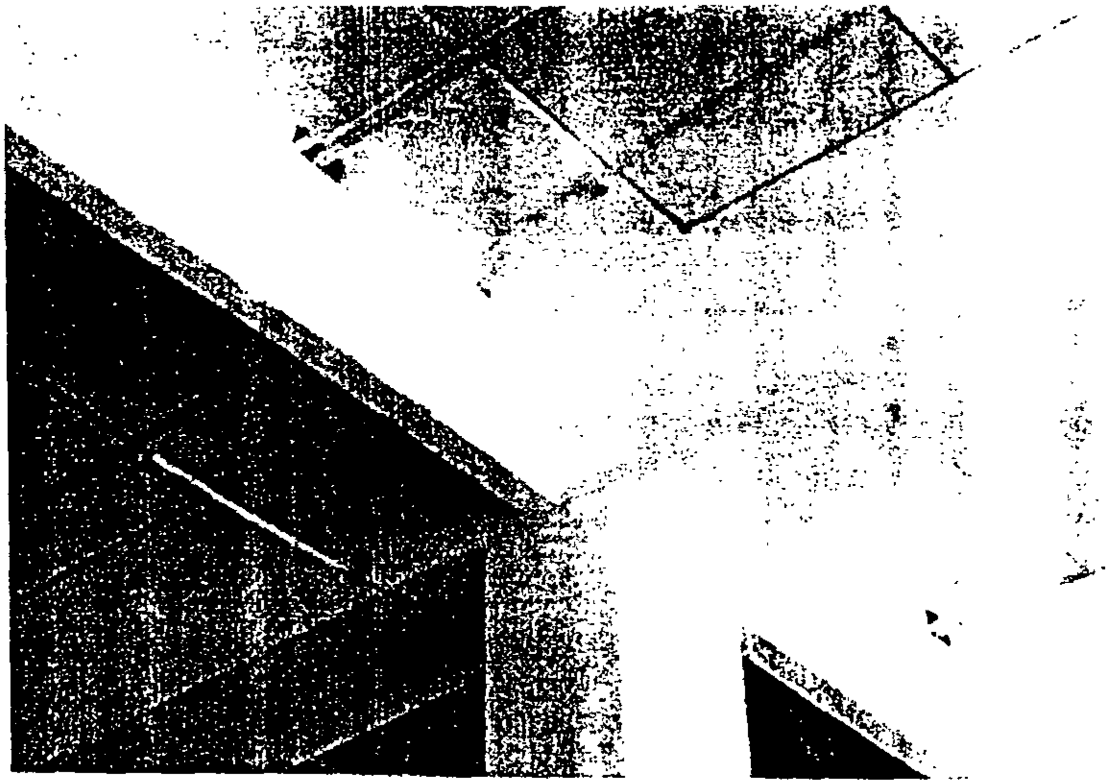

FOTO No. 5.- DETALLE DE CONEXION PROPUESTA. OBRA: HIPERMERCADO AUCHAN - TLATELOLCO, CD. DE MEXICO

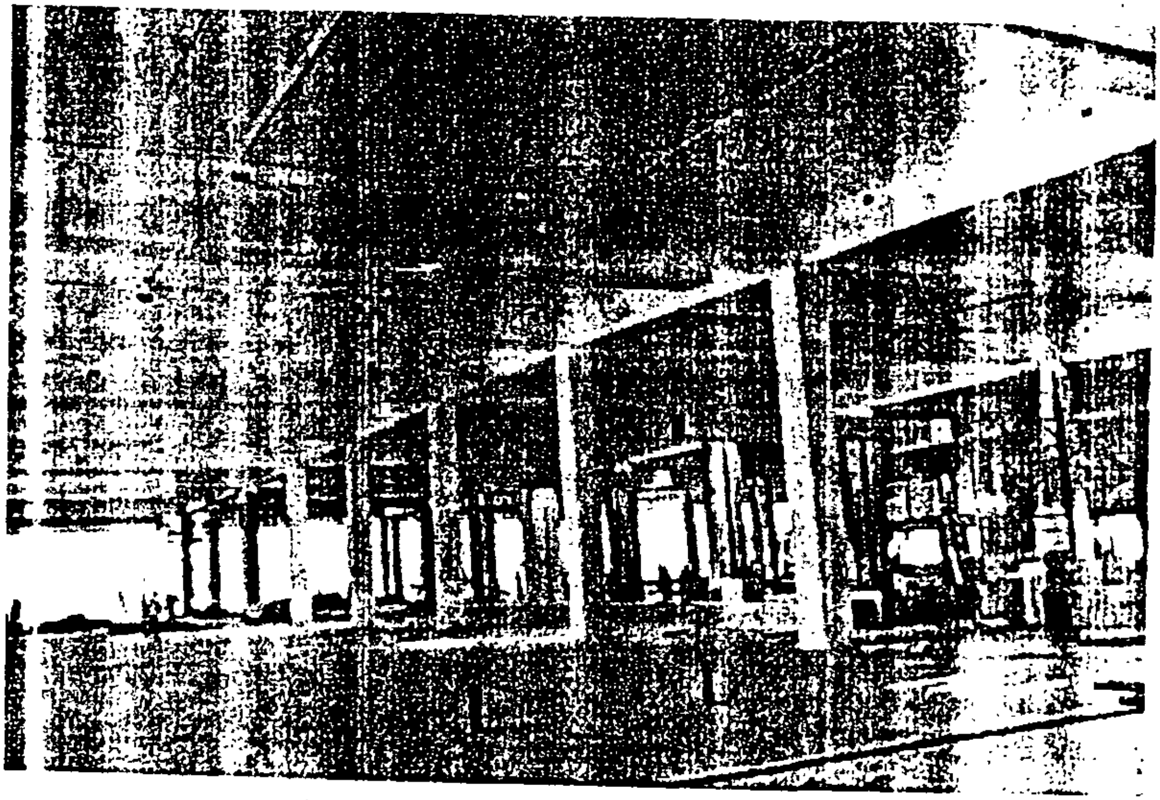

FOTO No. 6.- VISTA INTERIOR DE HIPERMERCADO AUCHAN-TLATELOLCO, CD. DE MEXICO ya que en esta etapa las columnas trabajan como péndulos invertidos y puede ser muy peligroso para la estabilidad de la estructura si se tienen desplazamientos grandes. También se recomienda en estos casos, en la medida de lo posible, colar los nodos de los niveles inferiores antes de continuar con el montaje de los niveles superiores.

Tabla No. 1 .- Comparativa de grupo de conexiones mas usadas :

TIPO DE CONEXIÓN	FABRICACIÓN	TRANSPORTE	MONTAJE	REQUERIMIENTOS	APARIENCIA
MÉNSULA CORTA	Complicada, sobre todo cuando se tienen ménsulas en las cuatro direcciones	Poco eficiente, se reduce el número de columnas por viaje	Se complica un poco por inclinaciones al momento de izar las columnas	Un mayor número de moldes, cortar en diferentes partes los moldes. Requiere soldadura para armado de trabes y columnas, también soldadura de campo	Ménsulas visibles en la parte inferior de las trabes, placas de acero y soldadura visibles
MÉNSULA LARGA	Muy complicada, sobretodo cuando se tienen ménsulas en las cuatro direcciones	Complicado, se minimiza el número de columnas por viaje y se necesitan accesorios especiales para su transporte	Complicado debido a la geometría de las columnas son poco manejables	Un mayor número de moldes y cortarlos en diferentes partes. Requiere soldadura en armados y de campo	Junta visible en unión con trabes, soldadura y placas visibles
POSTENSADO	Se complica al requerirse gran precisión para fijar los ductos en el armado de trabes y columnas.	En caso de que las columnas no cuenten con ménsulas se simplifica	En las columnas se simplifica cuando no llevan ménsulas, pero se requiere gran precisión para hacer coincidir los ductos de trabes y columnas, lo cual retrasa los tiempos de ejecución	Personal especializado para hacer los trabajos de postensado. En caso de contar con ménsulas se requiere apuntalamiento temporal	Juntas visibles en unión de columnas y trabes.
CONEXIÓN PROPUESTA	Se simplifica, se puede dar un mayor número de usos a los moldes al no tener que cortarlos,por lo que se requiere un menor número No es necesario fijar ningún tipo de accesorio al armado de las trabes y columnas	Se optimiza al poder transportar una mayor número de columnas por viaje	Se simplifica mucho. No se presentan problemas de inclinación al izar las columnas, se optimiza los tiempos de ejecución	NO se necesita personal especializado ni soldadura, se pueden emplear fierreros y albañiles que estén trabajando en la cimentación.	Como si fuera una conexión monolítica, NO quedan visibles ningún tipo de junta

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CURSOS ABIERTOS

XXVI CURSO INTERNACIONAL DE INGENIERIA SÍSMICA

MOUDLO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

REPARACION DE EDIFICIOS DAÑADOS POR EL TERREMOTO DE 1985 EN MEXICO

EXPOSITOR: M. EN C. ENRIQUE DEL VALLE CALDERON
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SEPTIEMBRE DEL 2000

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REPARACION DE EDIFICIOS DAÑADOS POR EL TERREMOTO DE 1985 EN MEXICO.

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REPARACION DE EDIFICIOS DAÑADOS POR EL TERREMOTO DE 1985 EN MEXICO

Resumen

Se discuten los criterios empleados en la reparación de algunos edificios dañados por el terremoto del 19 de septiembre de 1985 en la Ciudad de México, y se presentan dos ejemplos de aplicación.

Se presentan también los resultados obtenidos en las pruebas de vibración forzada efectuadas en dos edificios reparados antes de dicho sismo y que se comportaron satisfactoriamente en 1985, explicando las razones de ese buen comportamiento.

Introducción

Después de un temblor intenso un número importante de edificios queda -- con daños importantes; en muchos casos estos daños pueden considerarse aceptables de acuerdo con la filosofía de diseño sísmico vigente en la mayoría de los reglamentos de diseño por sismo, que recomiendan el uso de fuerzas reducidas en términos de la ductilidad esperada en la estructura, con respecto a la respuesta elástica que presentaría la misma estructura si fuera sometida a un sismo similar al empleado para diseñarla.

La base de esta filosofía es el comportamiento inelástico de la estructura cuando los esfuerzos en algunas secciones críticas son mayores que los de diseño, desarrollándose articulaciones plásticas que modifican el comportamiento de la estructura, reduciendo su rigidez original. Se recomienda que las articulaciones plásticas se formen en las vigas y no en las columnas u otros elementos resistentes verticales (refs. 1, 2) con objeto de evitar demandas de ductilidad local excesivas que puedan conducir a colapsos indeseables. Es conveniente también un buen detallado de los armados para garantizar que no ocurrirán fallas prematuras de tipo frágil. Se recomienda asimismo que el período de la estructura sea distinto de los períodos dominantes en el movimiento del suelo, (ref. 1) para evitar problemas de resonancia. Esto implica que estructuras rígidas, con períodos cortos pueden ser más vulnerables en terrenos firmes que tienen períodos dominantes pequeños (de menos de 0.5 seg.) durante movimientos sísmicos, y que las estructuras de altura media con períodos de vibración entre 1 y 2 seg., son más susceptibles a sufrir daños cuando se desplantan sobre terrenos blandos en los que los períodos dominantes durante los movimientos sísmicos pueden ser de ese orden.

Considero que esa filosofía de diseño sísmico es satisfactoria únicamente - para ciertos tipos de estructura construídos en determinado tipo de terreno. Si se observan los espectros de respuesta típicos para terrenos blando y duro (fig. 1) puede concluirse que la ductilidad de la estructura es benéfica en el caso de terrenos duros, porque cuando la estructura pierde rigidez debido a la formación de articulaciones plásticas y su período se alarga, ref. 3, la respuesta será menos severa; sin embargo, en el caso de terrenos blandos, que tienden a producir movimientos casi armónicos con un gran pico de respuesta para estructuras cuyos períodos son cercanos al período dominante en el movimiento del suelo, puede haber dos situaciones: la respuesta de la estructura disminuirá cuando su período fundamental de vibración se alargue si el período original es mayor que el período dominante en el movimiento del suelo, pero dicha respuesta se incrementará al alargarse el período, si su valor original es menor que el dominante en el suelo, aunque este incremento puede ser menor que el correspondiente al comportamiento perfectamente elástico porque las ordenadas espectrales de respuesta se reducirán en función del amortiguamiento adicional debido al comportamiento inelástico.

Lo anterior fue claramente demostrado en el sismo del 19 de septiembre de 1985 en la zona de terreno blando de la ciudad de México, donde muchos edificios de altura media, entre 6 y 15 pisos, ref. 4, fueron severamente afectados o colapsaron, mientras que edificios similares en terrenos duros o de transición no fueron afectados.

Un buen número de edificios incluídos en ese intervalo de alturas están siendo reparados actualmente. En este trabajo se describe el tipo de daño que sufrieron y los criterios de reparación empleados en varios casos en que el autor ha sido consultor.

Criterios de reparación

Todo edificio dañado que quede en pie después de un temblor intenso puede ser reparado, pero esto debe decidirse en función de la extensión de los daños, la posible inclusión de una estructura adicional, la confianza del propietario en el ingeniero estructurista y el costo de la reparación comparado con el costo de reposición del edificio, incluyendo la demolición.

Se han empleado distintos métodos para reparar estructuras e incrementar su resistencia y rigidez, refs. 5 y 6; los más usuales son el encamisado de columnas y vigas y/o la adición de muros de cortante o de diagonales de contraventeo en varias crujías. La eficiencia de estas soluciones es variable; sin embargo, en mi opinión, se debe dar atención especial a las características dinámicas de la estructura resultante, ya que en muchos casos los daños pueden atribuirse a condiciones cercanas a la resonancia en la estructura original, como en el caso de las estructuras antes citadas en que el período original es un poco menor que el período dominante en el movimiento del suelo, ya que si el criterio de reparación empleado no toma en cuenta las características de la estructura resultante puede quedar en peligro de sufrir daños más severos en movimientos futuros. En consecuencia, es importante tener una explicación razonable de las causas de las fallas.

Por ejemplo, después del sismo de 1985 el encamisado de columnas ha sido ampliamente usado en la ciudad de México para la reparación de edificios con sistemas de piso a base de losas planas aligeradas formando marcos equivalentes con las columnas. Esta solución no incrementa la rigidez ante fuerzas laterales de manera importante, porque dicha rigidez depende fundamentalmente de la rigidez relativa del sistema de piso, que suele ser muy baja comparada con la de las columnas en este caso; sin embargo, agrega en ocasiones masas considerables a la estructura original, por lo que el período fundamental de vibrar después de la reparación puede ser semejante al período original y si la estructura fue dañada por estar en condiciones cercanas a la resonancia la situación no se cambió y la resistencia adicional puede no ser suficiente para evitar daños futuros. Considero que es muy importante modificar las características dinámicas de la estructura si se sospe cha que los daños pueden atribuirse a condiciones cercanas a la resonancia. En este caso se logran soluciones más adecuadas agregando muros de rigidez o contravientos diagonales, los que usualmente proporcionan rigidez a fuerzas laterales suficiente para cambiar de manera importante las características dinámicas de la estructura.

Ejemplos de aplicación

Como se mencionó anteriormente, en este trabajo se presentan los criterios de reparación empleados en algunos edificios dañados, ubicados en la zona de terreno blando de la ciudad de México. Se discutirán dos casos.

El primero es un edificio de hospital localizado en una zona de máximos daños, con varios colapsos totales en un radio de 100m. El edificio tiene ocho pisos y sótano (figs. 2, 3, 4). La estructura original era a base de marcos de concreto reforzado y muros de concreto en algunas crujías, por lo que era relativamente rígido comparado con las propiedades del suelo; sin embargo, hubo fallas importantes en las columnas del sótano causadas -- por golpes contra la losa del piso de un estacionamiento anexo cuyo nivel era aproximadamente 50cm abajo del nivel de piso dentro del edificio (fig. 5); la cimentación tiene pilotes de control, fue necesario cambiar la mayoría de los puentes que transmiten la carga a los pilotes, pues se dañaron a causa del golpeteo. En el resto del edificio los daños fueron menores, con algunas losas de piso agrietadas, lo que se atribuyó a efectos de fuerza cortante.

Las normas de emergencia publicadas el 18 de octubre de 1985, que se emplearon para la reparación, pedían la reestructuración de cualquier edificio que hubiera sufrido daños mayores. El nuevo coeficiente sísmico era 67% mayor que el del reglamento en vigor antes del sismo, por lo que, para lograr que la estructura cumpliera con las nuevas normas fue necesario reforzar también los niveles superiores del edificio. Las columnas del sótano fueron encamisadas y algunos tramos del muro de retención, que no llegaban hasta el nivel de planta baja para dar iluminación al sótano, fueron cerrados para tener mayor capacidad resistente. Se agregaron también varios muros de concreto reforzado en los pisos superiores para satisfacer las demandas de resistencia de las normas de emergencia, estos muros se anclaron a los marcos empleando tramos de varillas que se alojaban en taladros y se rellenaban con resinas epoxy (fig. 6).

Las grietas de las losas de niveles superiores fueron rellenadas con resinas epoxy, y para aumentar su resistencia al corte se aplicó presfuerzo a las losas en la dirección longitudinal del edificio (fig. 7). No fue necesario incrementar el número de pilotes de la cimentación. La losa de estacionamiento que causó el problema se demolió a lo largo de su contacto con el edificio, dejando una trinchera para absorber movimientos futuros.

Se midieron los períodos de vibración en condiciones ambientales después de la reparación, obteniendo valores de 0.62 seg. en dirección transversal y 0.53 seg. en dirección longitudinal. Estos valores concuerdan razonablemente bien con los calculados. Se considera que estos períodos garantizan que el edificio tendrá una respuesta baja en movimientos futuros ya que están suficientemente lejos de los períodos dominantes del movimiento del suelo en ese lugar; asimismo, la causa principal del daño (la losa del estacio namiento) fue eliminada

El segundo caso corresponde a un edificio del servicio postal, el cual tiene dos cuerpos separados por una junta constructiva, uno de cinco niveles (con sótano) y el otro con diez niveles (incluyendo dos sótanos); la altura de ambos cuerpos es igual, ya que los entrepisos del primero tienen 7.20 m de alto, mientras que los del segundo cuerpo tienen sólo 3.60m. La separación entre los cuerpos es del nivel de planta baja a azotea (fig. 8), pues están unidos en planta baja y sótano. El primer cuerpo tiene planta rectangular de 90m por 40m, con dos claros de 5m y ocho de 10m en la dirección larga y dos claros de 8m y dos de 12m en dirección corta, (fig. 9). La estructura es de concreto reforzado, con losas planas aligeradas de 50cm de espesor total, formando marcos equivalentes con las columnas.

El problema fundamental en este cuerpo fue provocado por una gran marquesina en voladizo alojada a la mitad de la altura del segundo entrepiso en el marco longitudinal norte, la que estaba apoyada en una viga de torsión que reducía a la mitad la altura de las columnas de este marco (fig. 10) incrementando notablemente su rigidez comparada con la de los otros marcos longitudinales. Esto atrajo cortantes muy elevados hacia el marco norte durante el sismo de 1985 y provocó efectos torsionantes importantes. Cabe señalar que aparentemente la marquesina no estaba incluída en el proyecto original, habiendo sido añadida tal vez como modificación durante la construcción, pero sin tomar en cuenta los efectos desfavorables que provocaría en la respuesta sísmica del edificio. Casi todas las columnas del segundo piso en el marco norte fallaron durante el sismo de 1985 (fig. 11).

El otro cuerpo tiene planta más pequeña pero es irregular en los primeros niveles y tiene un atrio de planta baja a azotea en otra zona, lo que provocó problemas importantes por asimetría. Este cuerpo había sido dañado por un sismo previo y se habían adicionado dos muros de rigidez de concreto; sin embargo, este refuerzo no fue suficiente y el sismo de 1985 provocó daños importantes, con fallas en vigas y columnas en la zona del atrio y daño en muros no estructurales. Hubo golpeteo entre ambos cuerpos.

Se decidió reestructurar ambos cuerpos, usando elementos diagonales de acero. Las figs. 12 a 14 muestran la planta y elevación de la solución propuesta para el cuerpo grande. Se recomendó eliminar la marquesina y la viga de torsión que la soportaba. Esto ya se llevó a cabo eliminando la causa principal de los problemas de asimetría en este cuerpo. (fig. 15).

Se contraventearon ocho crujías de 10m en la dirección longitudinal, acopladas en parejas, en los marcos de fachada norte y sur y en la dirección corta se contraventearon doce crujías de 8m en marcos interiores. La rigidez se incrementó notablemente en ambas direcciones. Los períodos del modo fundamental calculados para la estructura original son 2.86 seg y 3.13 seg. en las direcciones longitudinal y transversal respectivamente; con los contravientos se reducen a 0.79 y 0.97 seg. Se considera que los problemas en este edificio no pueden atribuirse a condiciones cercanas a la resonancia ya que los períodos originales eran suficientemente largos y estaban en la rama descendente del espectro de respuesta. Los períodos garantizan una respuesta moderada.

Si la rigidización hubiera sido menor se correría el riesgo de dejar al edi ficio en situación peligrosa, pues sus períodos serían más cercanos a los del movimiento del terreno en el sitio y la respuesta se incrementaría.

Las columnas que enmarcan las crujías contraventeadas se reforzaron con ángulos de acero y celosías, ya que las cargas axiales que deben soportar son mayores que las que actuaban en ellas antes de la rigidización. Las conexiones entre la estructura original y los elementos de contraventeo son a base de placas y pernos de anclaje. No se requirió reforzar la cimentación con pilotes adicionales ya que el diseño original fue hecho para cargas vivas elevadas (del orden de 1 ton/m^2) y el nuevo proyecto considera cargas vivas para oficina únicamente.

Los trabajos de rigidización no se han iniciado aún por problemas económicos. No se dan detalles de la reparación propuesta para el otro cuerpo por falta de espacio. Es similar a la descrita.

Comentarios adicionales

Una innovación en ingeniería sísmica es la adición de amortiguamiento externo a las estructuras para absorber energía y reducir su respuesta. Se ha demostrado en numerosos artículos técnicos los efectos benéficos del amortiguamiento para reducir la amplificación de los movimientos del terreno; la ventaja fundamental de estos nuevos sistemas es que no es necesaria la formación de articulaciones plásticas para que se reduzca la respuesta, ya que los amortiguadores pueden diseñarse de tal manera que empiecen a actuar antes que la estructura llegue al comportamiento inelástico.

Recientemente se han propuesto dos tipos de amortiguador diferentes, (refs. 7, 8), uno emplea capas de material viscoelástico en contacto con placas de acero para absorber la energía por deformación; el otro usa balatas de freno para disipar energía por fricción contra placas de acero. Ambos pueden introducirse en crujías contraventeadas.

La empresa Pall Dynamics, de Canadá, de común acuerdo con el suscrito, ha presentado una propuesta para emplear sus dispositivos friccionantes en el edificio postal lo que conduce a ahorros importantes en las cantidades de acero necesarias para reparar la estructura, ya que las diagonales se diseñan para trabajar a tensión únicamente y los refuerzos de las columnas son más ligeros debido a que las cargas en ellas se reducen en virtud del amortiguamiento adicional. Esto hace que también en ocasiones no sea necesario reforzar las cimentaciones. El costo de los amortiguadores se cubre con las reducciones de acero de refuerzo, quedando una solución más económica y con un mejor comportamiento ante sismos futuros. La posibilidad de resonancia se reduce considerablemente con esta solución, pues el período cambia al ceder los amortiguadores.

La Universidad de California en Berkeley ensayó recientemente un modelo de edificio de acero de nueve niveles equipado con amortiguadores de fricción (fig. 16) en la mesa vibradora instalada en Richmond, con excelentes resultados. El informe, por Kelly, será publicado próximamente.

También Hanson, en la Universidad de Michigan ha estudiado los efectos de amortiguamiento suplementario en los edificios (ref. 9), con resultados muy promisorios.

Pruebas en edificios reparados

El excelente comportamiento de edificios rigidizados con diagonales de contraventeo durante el sismo de septiembre de 1985 en la Ciudad de México, pudo apreciarse en tres edificios de mediana altura que fueron reparados empleando este sistema tras haber sido dañados por temblores anteriores a 1985, (refs. 10 y 11), y que se comportaron muy satisfactoriamente en esta ocasión, (figs. 17 y 18). Con el patrocinio de la Fundación Nacional de Ciencias de los Estados Unidos, dos de estos edificios fueron estudiados dentro de un proyecto de investigación encabezado por Douglas Foutch de la Universidad de Illinois y yo mismo. En enero de 1987 llevamos a cabo pruebas de vibración forzada en ambos edificios empleando los vibradores de masas excéntricas propiedad del Instituto Tecnológico de California, y varios alumnos realizaron el análisis elástico e inelástico paso a paso de distintos modelos matemáticos de los edificios. La interpretación de los resultados de las pruebas de vibración forzada y su reconciliación con los de los modelos matemáticos se resumen en las figuras 19 y 20, (ref. 12).

Como puede verse en dichas figuras, una parte importante de los desplazamientos laterales se debe a rotación de la base, debido a las características del subsuelo blando de la Ciudad de México. Esto disminuyó un poco la eficiencia del sistema de contraventeo, pues, por ejemplo, en el edificio de Durango el período con base empotrada sería de 0.90 seg. mientras que el período con interacción es de 1.26 seg.; sin embargo, la solución funcionó bastante bien como se indicó anteriormente, ya que cambió radicalmente el período que tenía la estructura antes de la rigidización, de 1.86 seg., sacándola de una condición cercana a la resonancia, que fue la que le provocó los daños en un temblor mucho menos intenso que el de 1985. Para el edificio de Parque España la situación es similar. Se puede afirmar que si no se hubieran rigidizado y reforzado estos edificios hubieran colapsado en 1985.

Conclusiones y recomendaciones

La reestructuración de edificios puede ser exitosa si se toman en cuenta una serie de detalles, como verificar que las propiedades dinámicas de la estructura resultante sean adecuadas con respecto a las características del movimiento del terreno; el flujo de fuerzas entre la estructura original y los refuerzos debe estudiarse cuidadosamente para conectarlos correctamente Con frecuencia es necesario reforzar las columnas que enmarcan las crujías contraventeadas debido a las fuerzas adicionales que atraen, en virtud de la redistribución de efectos sísmicos causados por la mayor rigidez de estas crujías. Es necesario verificar también que la cimentación tenga la resistencia y rigidez suficientes para absorber adecuadamente la nueva distribución de fuerzas. También es muy importante asegurarse de que los diafragmas horizontales tengan la rigidez y resistencia suficientes para transmitir las fuerzas a los nuevos elementos; si no es así, es necesario reforzarlos.

El empleo de amortiguadores para reducir la respuesta durante temblores intensos constituye una técnica muy promisoria que probablemente sea empleada pronto en la reestructuración de edificios dañados y en la construcción de edificios nuevos.

Reconocimiento

El diseño detallado del edificio uno fue llevado a cabo en la oficina del Ing. Enrique Calleja; el del edificio dos en la oficina de los Ingenieros García Jarque, en ambos casos con mi asesoría. Amablemente prepararon parte de las ilustraciones de este trabajo y proporcionaron información adicional. Roberto Meli leyó el borrador original e hizo valiosas sugerencias.

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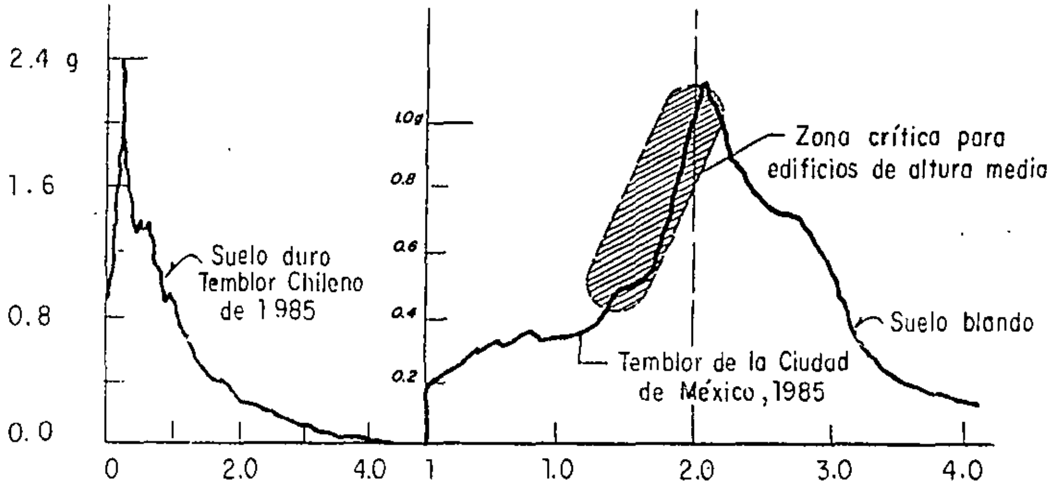


Fig 1 Espectros de respuesta típicos para terrenos duros y blandos, para 5 % de amortiguamiento y comportamiento elástico.

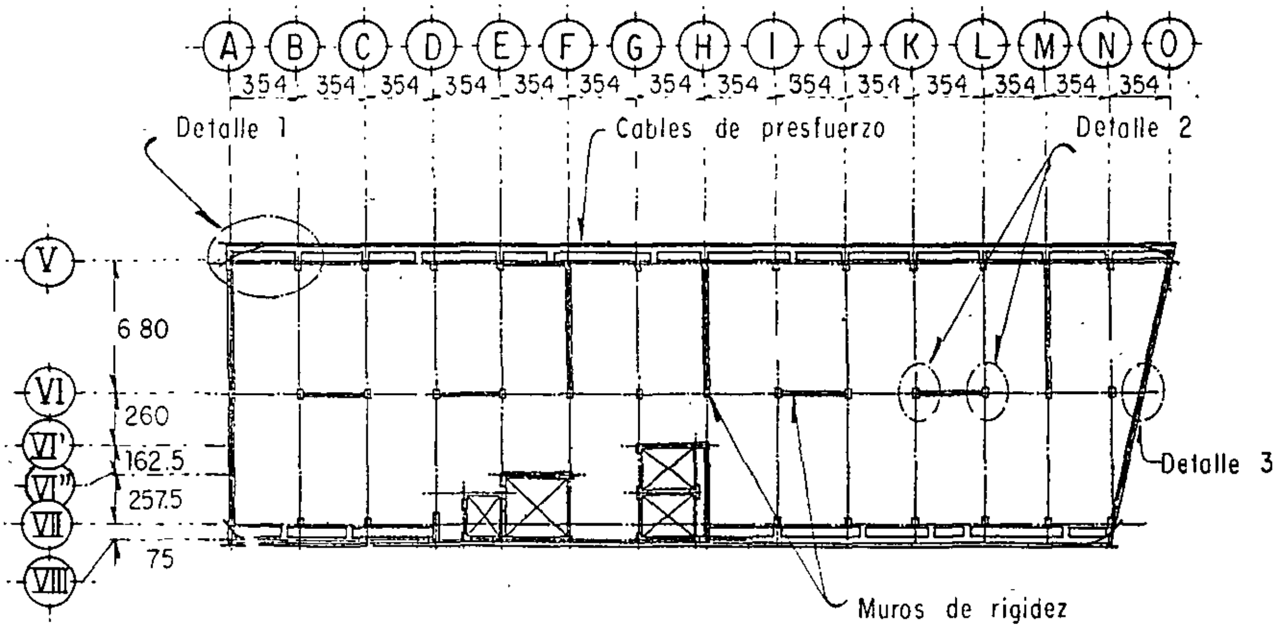


Fig 2 Planta tipo . Hospital

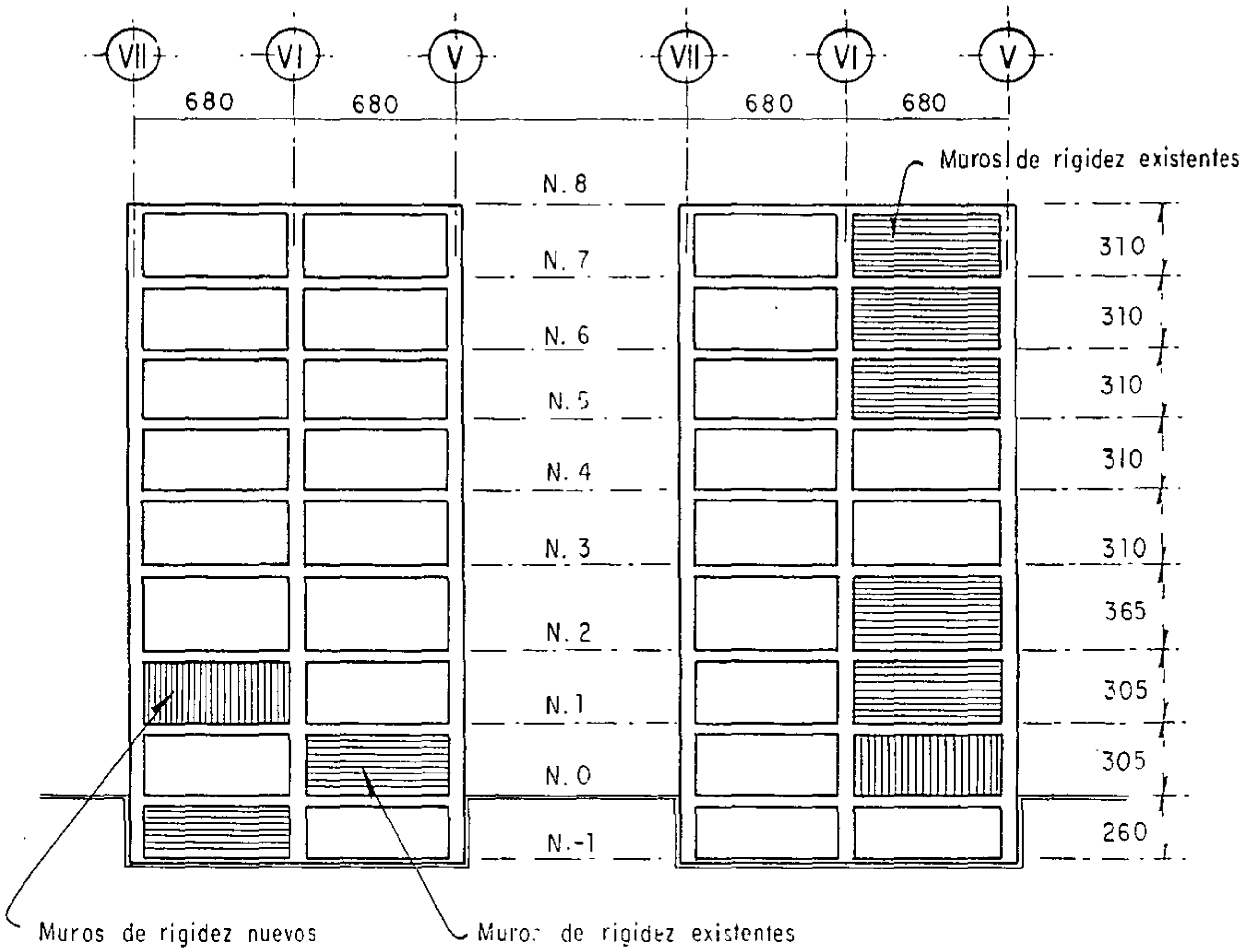


Fig 3 Elevación de marcos J y N . Hospital

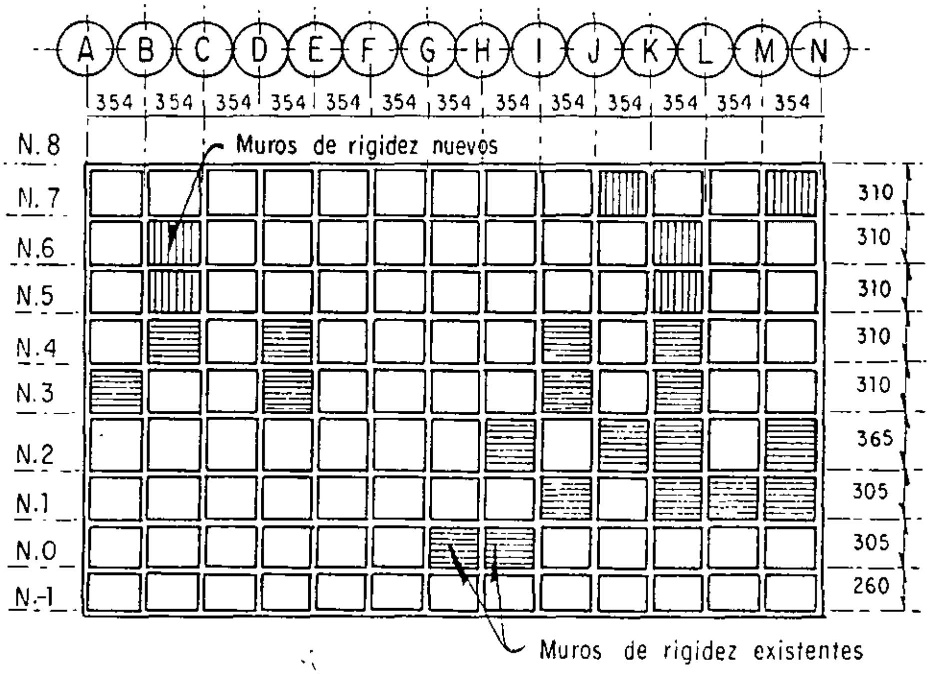


Fig 4 Elevacion del marco VI , Hospital

Fig 5 Losa de estacionamiento que causó los daños

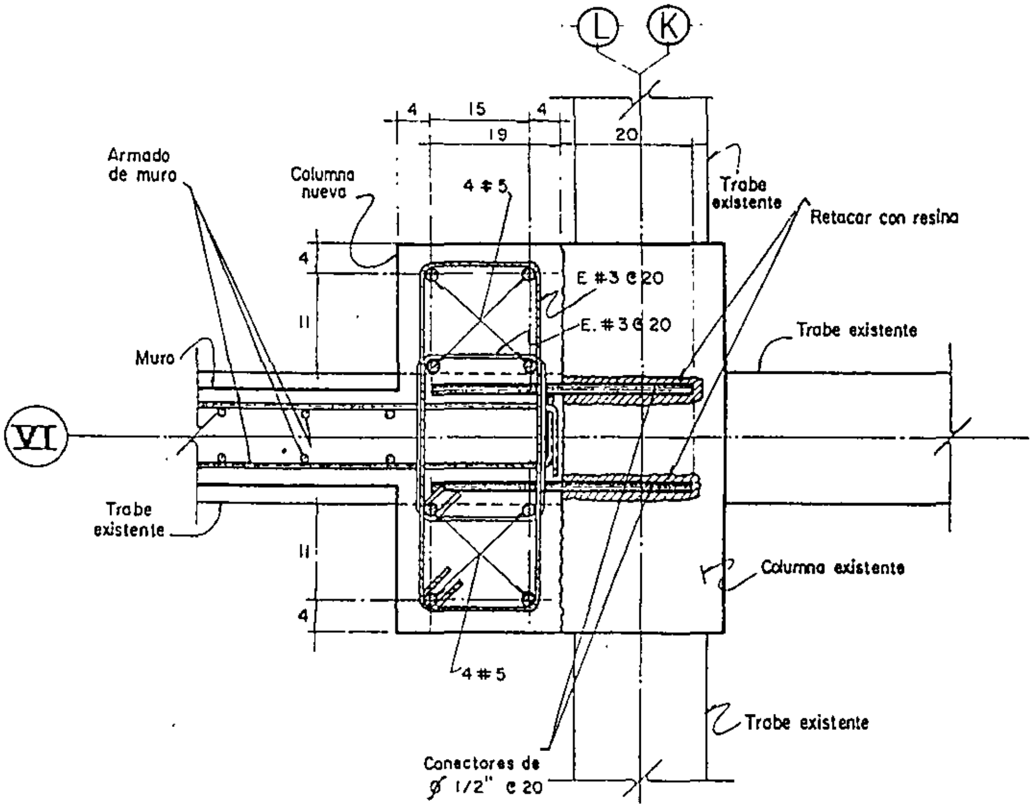
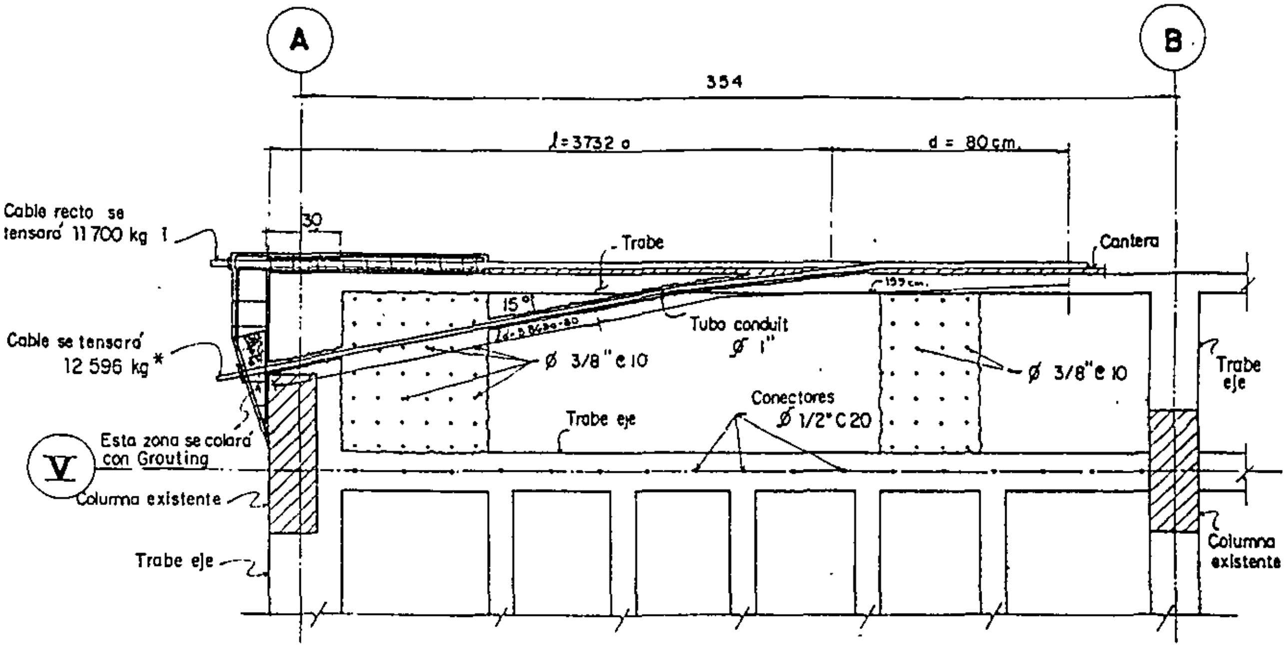


Fig 6 Detalle Nº 2 (Ver fig. 2). Hospital



* Area de torón
A = 920 cm²
fs = 18 750

P L A N T A

Fig 7 Detalle Nº 1 (Ver fig 2). Hospital

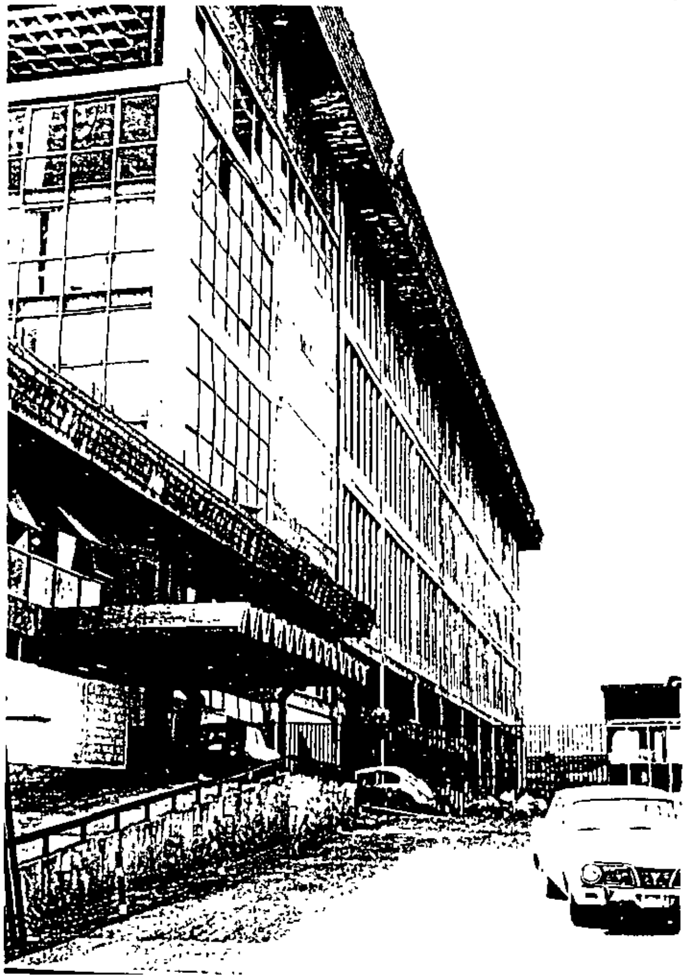


Fig 8 Edificio postal

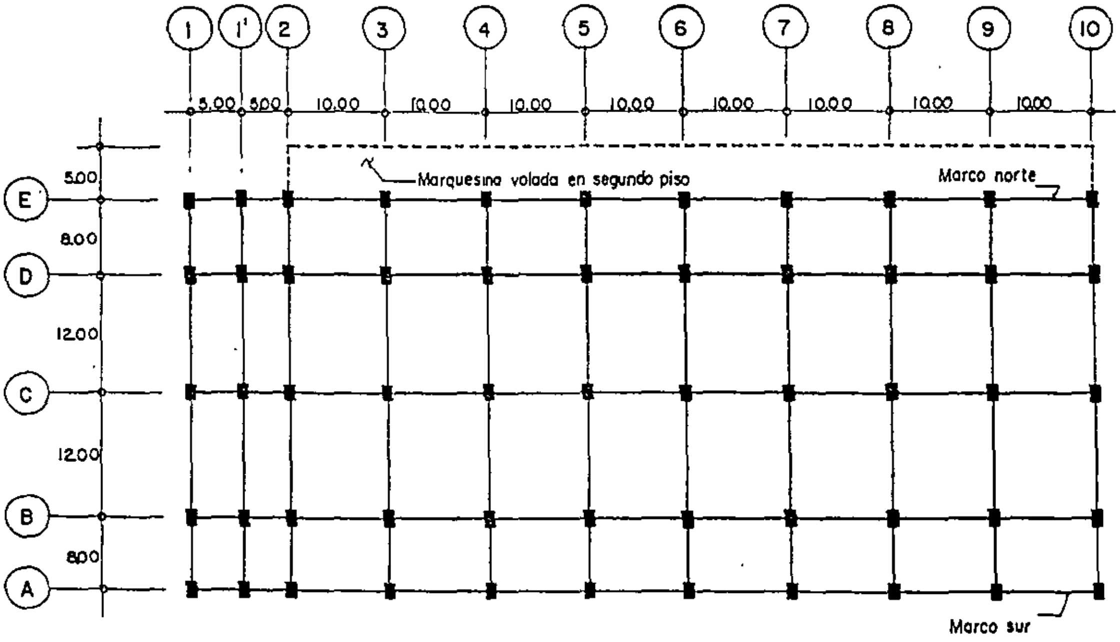


Fig 9 Planta típica. Edificio postal

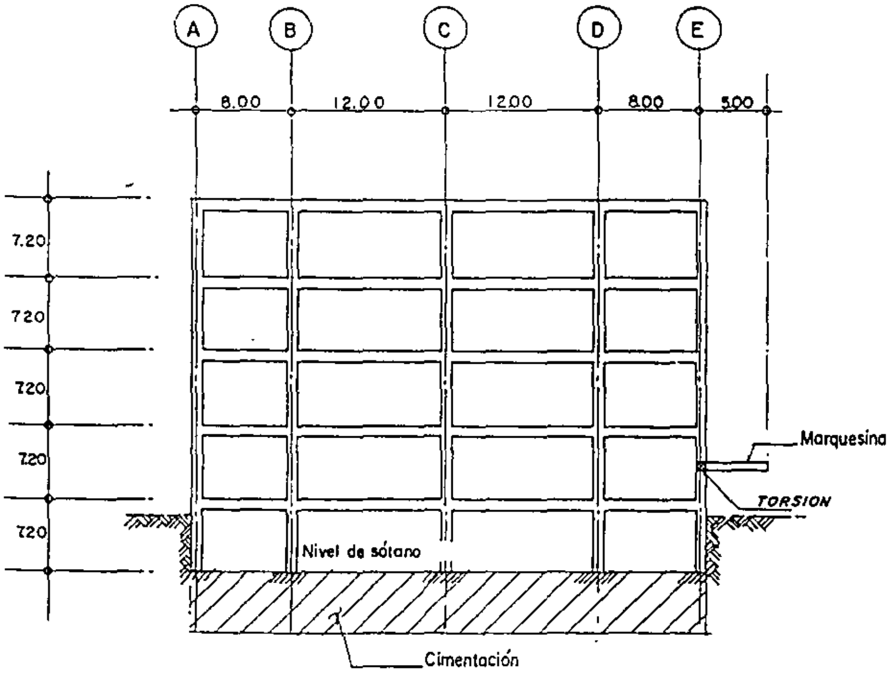


Fig 10 Marcos transversales. Edificio postal

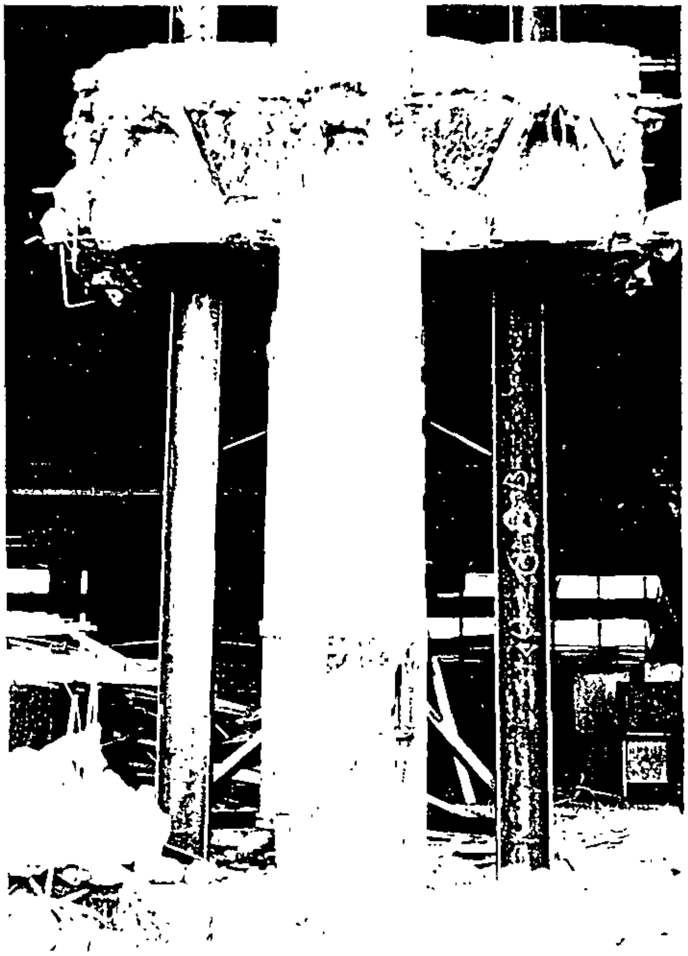


Fig 11 Daño en columnas

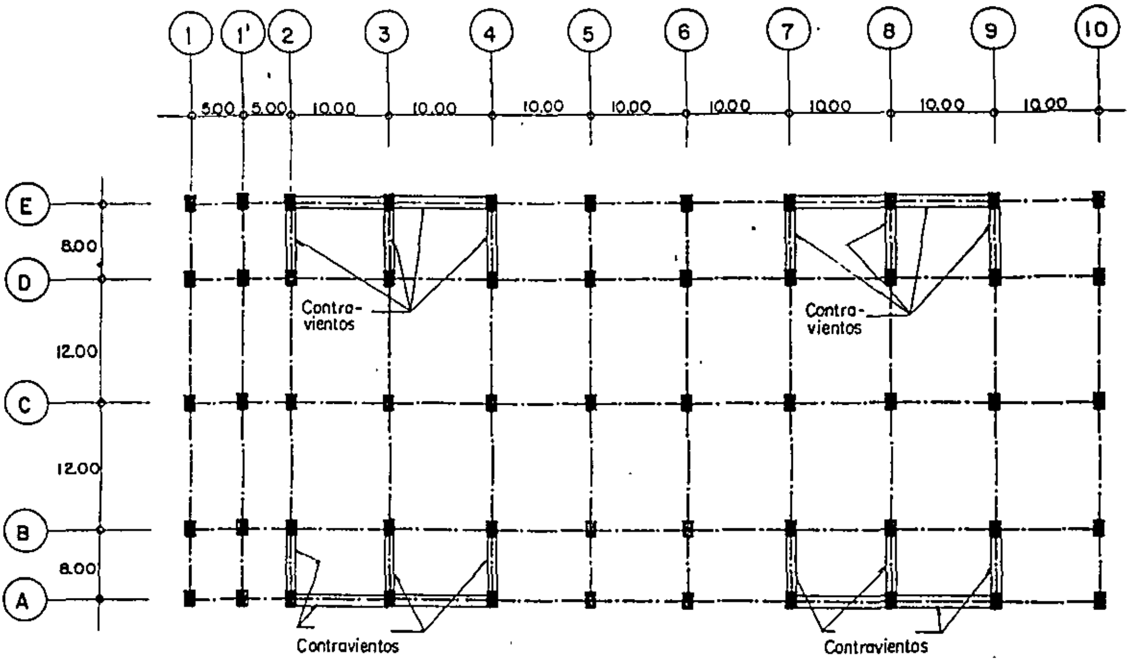


Fig 12 Localización de contravientos de refuerzo. Edificio postal

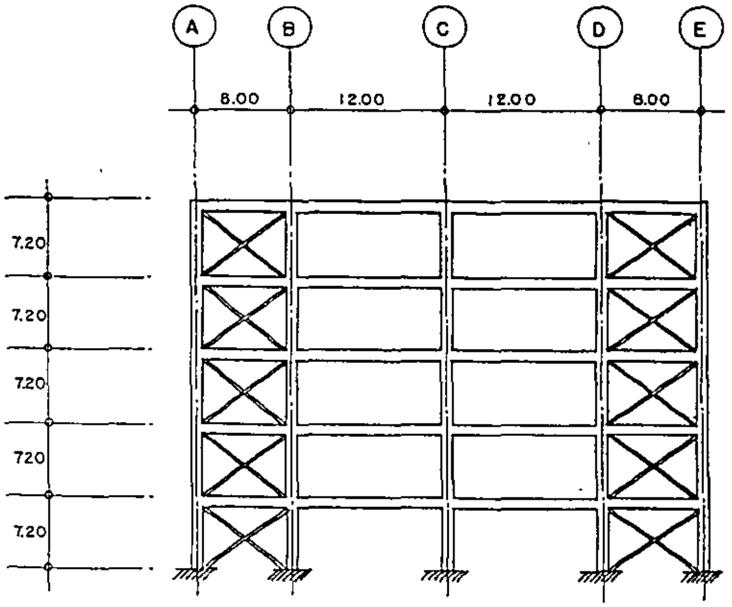


Fig 13 Marcos transversales con contravientos
(Edificio Postal)

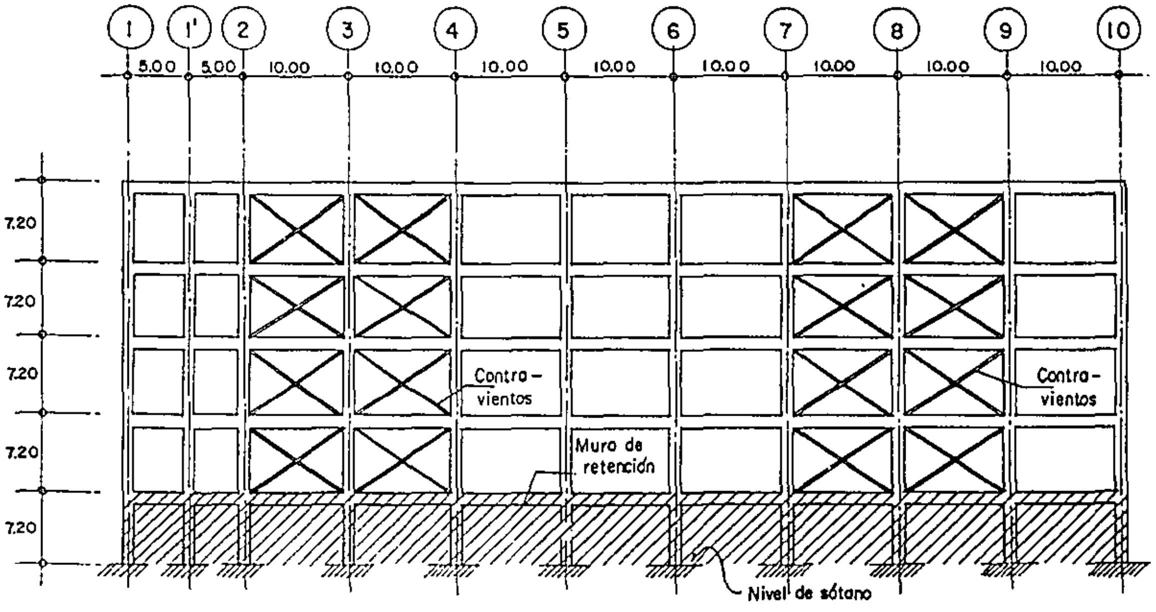


Fig 14 Elevación de marcos norte y sur con contravientos (Edificio Postal)

Fig 15 Demolición de marquesina

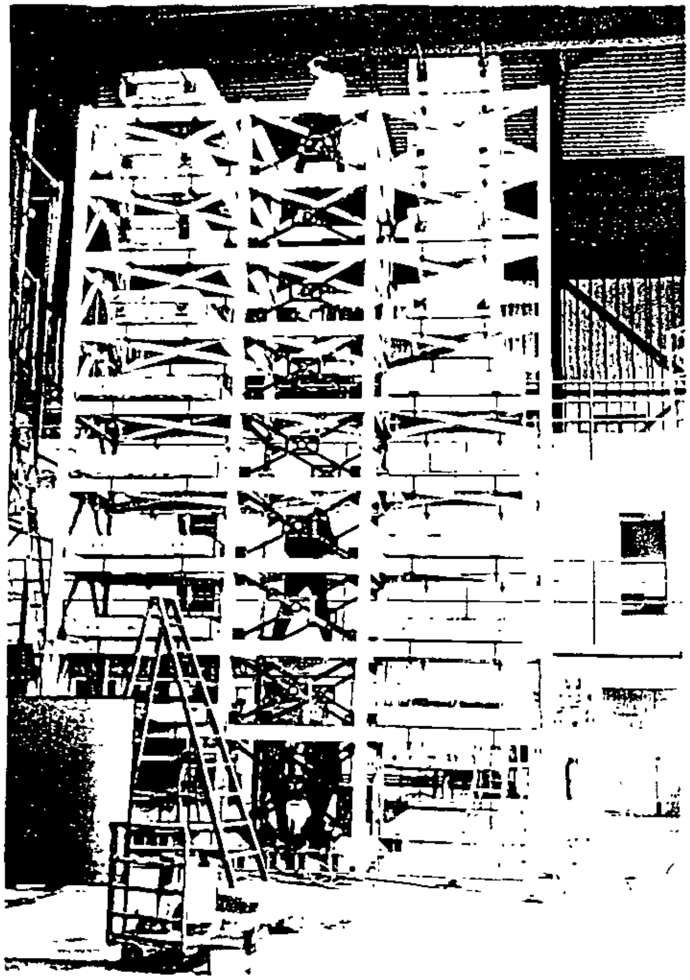
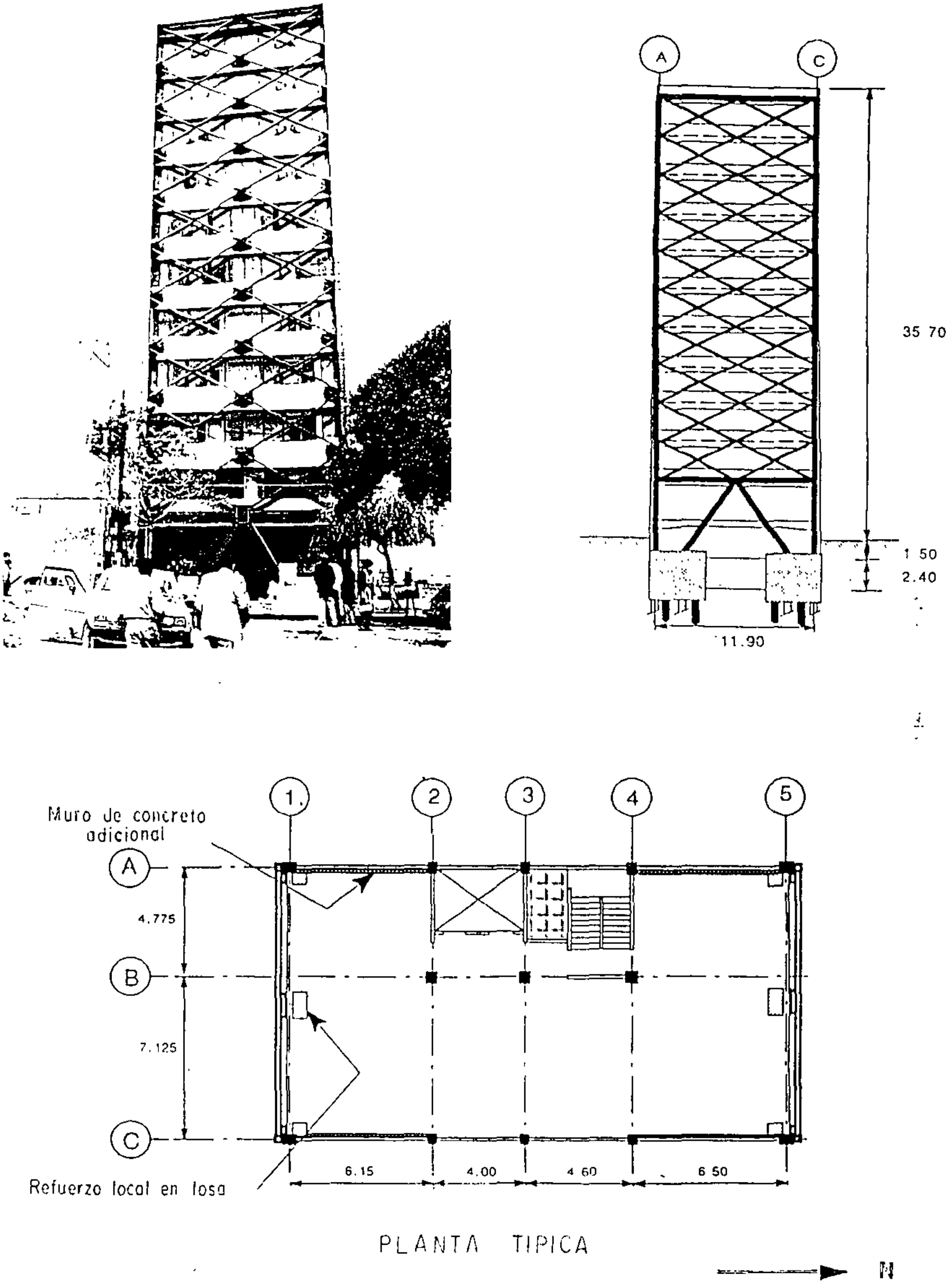
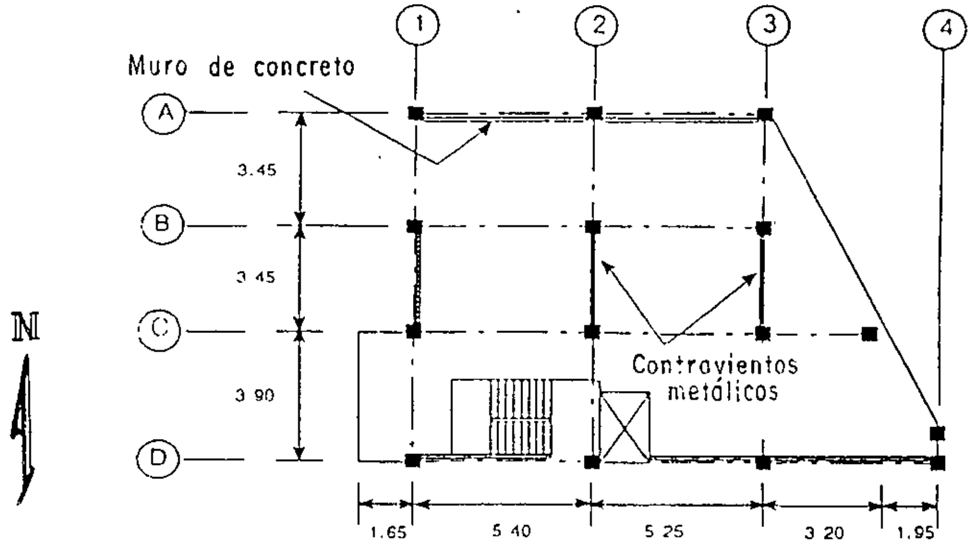


Fig 16 Marco con amortiguadores



PLANTA TIPICA

Fig 17 Edificio en calle Durango



PLANTA TIPICA

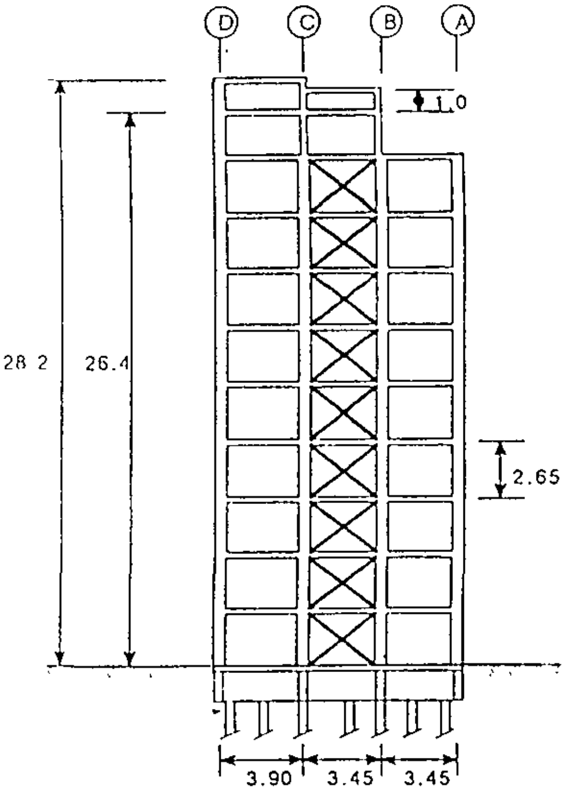
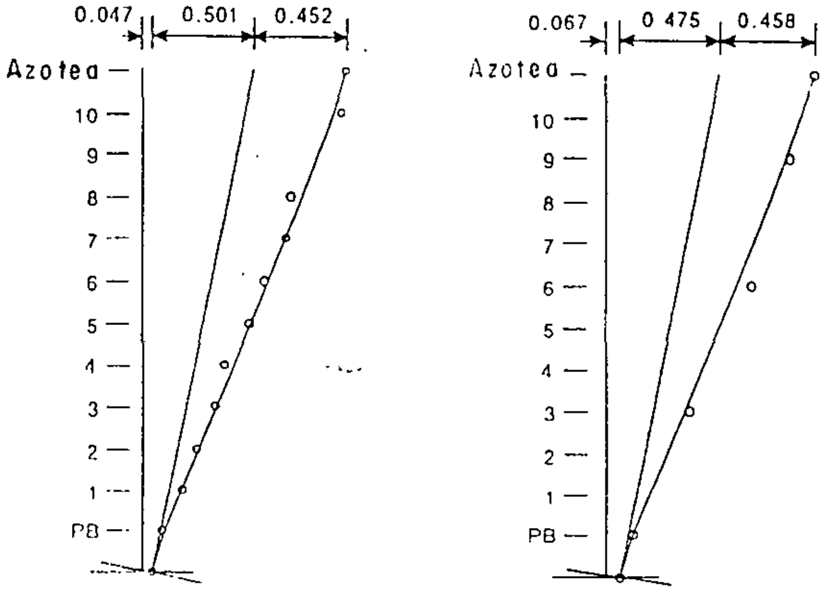


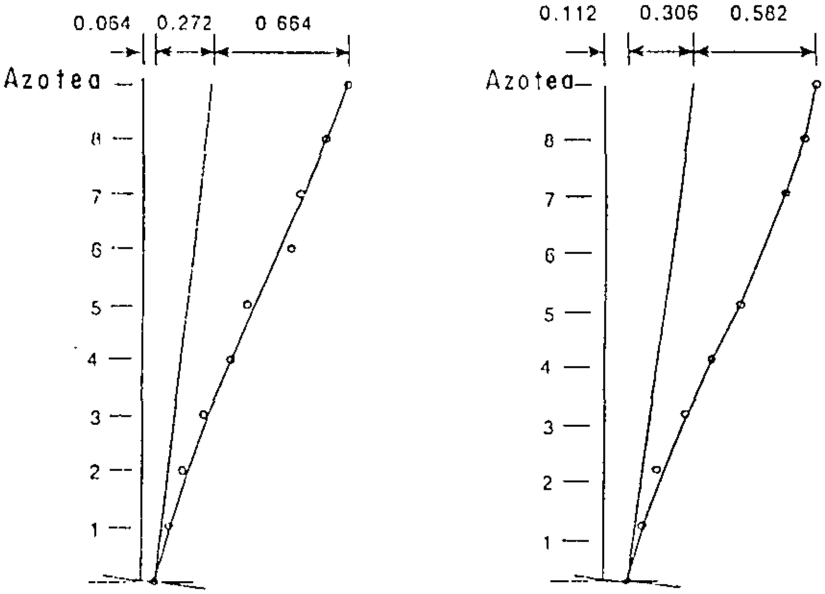
Fig 18 Edificio reparado con diagonales de acero,
(Ubicado en Parque España)



Forma del primer modo E - W
Período : 1.26 seg.
Amort. : 2.8 %

Forma del primer modo N - S
Período : 0.99 seg.
Amort. : 4.8 %

Fig 19 Formas modales para el Edificio en Calle Durango



Forma del primer modo N-S
Período : 1.11 seg.
Amort. : 3.0 %

Forma del primer modo E- W
Período : 0.90 seg.
Amort. : 6.9 %

Fig 20 Formas modales para el Edificio en Parque España

FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

XXVI CURSO INTERNACIONAL DE INGENIERIA SÍSMICA

MOUDLO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

REFORZAMIENTO POST – SISMICO DESPUES DEL TERREMOTO DE MEXICO EN 1985

EXPOSITOR: M. EN C. ENRIQUE DEL VALLE CALDERON
PALACIO DE MINERIA
SEPTIEMBRE DEL 2000

REFORZAMIENTO POST-SISMICO DESPUES DEL TERREMOTO DE MEXICO EN 1985

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RESUMEN

Se presentan las estadísticas de los daños causados por el sismo del 19 de Septiembre de 1985 en la Ciudad de México. Se comenta sobre la necesidad de hacer una evaluación antes de reparar un edificio, sobre las distintas técnicas de reforzamiento empleadas y se ilustran éstas con algunos ejemplos. Se hacen también comentarios sobre el comportamiento de algunas estructuras reforzadas antes del sismo y de la adición de dispositivos para absorber energía. Al final se presentan conclusiones y recomendaciones.

ABSTRACT

Statistics of damages caused of the September 19, 1985 earthquake are presented. Commentaries are made concerning the need of evaluation before making the repair project, as well as regarding the different techniques employed. Some examples are illustrated. Commentaries about the behavior of repaired structures and the addition of energy absorbing devices are made. Finally conclussions and recommendations are presented.

INTRODUCCION

El sismo del 19 de Septiembre de 1985 ha sido el más destructivo en la historia de la Ciudad de México. La magnitud del movimiento fue de 8.1 en la escala de Richter, el epicentro se ubicó a 400 km de la Ciudad. Al día siguiente se tuvo una réplica de magnitud 7.5 que afortunadamente ya no produjo muchos daños adicionales

Las intensidades en la Ciudad, en la escala de Mercalli modificada, variaron de VI en la periferia a valores de VIII, IX y X en algunas zonas de terrenos blandos, que es donde los efectos de los sismos se amplifican y causan los daños más importantes. En la zona de máximos daños colapsaron totalmente 133 edificios, 353 sufrieron colapsos parciales y 271 tuvieron daños graves, una buena parte de los cuales tenía de 6 a 15 niveles, lo que es reflejo de problemas dinámicos asociados a los periodos dominantes del terreno, que en esa zona de máximos daños varían entre 1.5 y 2.5 segundos, periodos que son cercanos a los de edificios flexibles, de mediana altura, quizá dañados por los primeros ciclos intensos del movimiento.

Esta situación puso en evidencia que la filosofía de diseño sísmico empleada en la mayoría de los Reglamentos de Diseño, que se basa en la reducción de los efectos sísmicos debido al comportamiento inelástico de las estructuras, empleando factores de reducción por ductilidad, puede no ser lo más adecuado para edificios de altura media desplantados en terrenos blandos

Si se observan los espectros de respuesta típicos para terrenos blando y duro (fig. 1) puede concluirse que la ductilidad de la estructura es benéfica en el caso de terrenos duros, por que cuando la estructura pierde rigidez debido a la formación de articulaciones plásticas y su periodo se alarga, la respuesta será menos severa; sin embargo, en el caso de terrenos blandos, que tienden a producir movimientos casi armónicos con un gran pico de respuesta para estructuras cuyos periodos son cercanos al periodo dominante en el movimiento del suelo, puede haber dos situaciones: la respuesta de la estructura disminuirá cuando su periodo fundamental de vibración se alargue si el periodo original es mayor que el periodo dominante en el movimiento del suelo (edificios altos), pero dicha respuesta se incrementará al alargarse el periodo, si su valor original es menor que el dominante en el suelo, aunque éste incremento será menor que el correspondiente al comportamiento perfectamente elástico por que las ordenadas espectrales de respuesta se reducirán en función del amortiguamiento adicional debido al comportamiento inelástico.

En éste último caso, emplear factores de reducción por ductilidad elevados puede conducir a la estructura a daños graves o colapso, por lo que es deseable usar valores bajos y, de preferencia, disipar la energía sísmica empleando dispositivos especiales para este fin.

CUANTIFICACION DE LOS DAÑOS EN LA CIUDAD DE MEXICO

La Tabla 1 compara el número de construcciones dañadas con el número de existentes en la zona más dañada de la Ciudad, tomando como variable el número de niveles. Para construcciones de uno a cinco niveles, el porcentaje de estructuras dañadas en relación con las existentes es del orden del 1.3%, debido en general a que son construcciones con periodos cortos alejados de los dominantes en la vibración del suelo; sin embargo, el aumentar el número de niveles, aumenta sensiblemente el porcentaje de daños, ya que para edificios de seis a ocho niveles se dañó, en la zona estudiada, el 8.4% de los edificios existentes, de nueve a doce niveles el 13.5% y de trece a quince niveles el 13.7%, bajando notablemente los daños para edificios más altos, cuyos periodos ya son mayores, en general, que los dominantes en la vibración del suelo

Cabe aclarar que si la estadística se toma considerando solo el número de construcciones dañadas, comparando los de una cierta cantidad de niveles con el total de dañadas, los porcentajes cambian radicalmente, pues en números absolutos hubo más daños en las de uno a dos niveles, que corresponden al 46% de las dañadas, siguiendo las de tres a cinco niveles con el 24% del total y las de seis a ocho niveles con el 18%, por lo que resultó necesario comparar con el número de construcciones existentes de cada tipo, para revelar el problema dinámico en toda su extensión.

Tabla 1.- Daños en edificios en función del número de niveles

No. de niveles	Edificios dañados gravemente	% del total de dañados	Edificios existentes en la zona estudiada	Edificios dañados entre edificios existentes.
1 a 2	346	45.7	37484	1.07%
3 a 5	179	23.6	13498	1.3%
6 a 8	136	18.0	1616	8.4%
9 a 12	72	9.5	531	13.5%
13 a 15	22	2.9	160	13.7%
16 a 18	0	0	22	0%
mas de 18	2	0.3	47	4.3%
Sumas	757	100.0	53358	1.4%

Al considerar los daños en cada uno de los sectores en que se subdividió la zona de máximos daños, se encontró que hubo sectores donde el porcentaje de daños en estructuras de seis a ocho niveles fue de 34% de las existentes, otros donde las más afectadas resultaron las de nueve a doce niveles, con 67% de las existentes y otros donde para estructuras de trece a quince niveles se afectaron el 60% y hasta el 75% de las existentes, lo cual seguramente fue causado por la variación de los periodos dominantes del suelo antes mencionada, ref. 1. Se pudo ver también que edificios idénticos a los dañados seriamente en una zona, no presentaban ningún daño cuando estaban construidos sobre terrenos más firmes, con características de vibración diferentes.

Cabe mencionar que el número total aproximado de construcciones en el área metropolitana era cercana a 1.9 millones y que según datos oficiales de fines de 1985, el número total de construcciones con daños graves en la Ciudad fue de 1628 y, si se incluyen otras construcciones que solo requieren reparación parcial, el total sube a 3382 edificaciones dañadas. En referencias posteriores, y después de haber hecho un levantamiento más completo (ref. 2) se mencionan 3820 estructuras hasta cuatro niveles y 1205 de más de cuatro niveles con algún tipo de daño, lo que da un total de 5025 edificaciones dañadas en toda la ciudad.

Como puede verse, con respecto al total de edificaciones, las dañadas representan un porcentaje muy bajo, lo que indica que a pesar de la gran intensidad del sismo, la ciudad en su conjunto tuvo un comportamiento muy aceptable; pero como se mencionó, hubo una gran selectividad en las estructuras afectadas, principalmente en cuanto al número de niveles y terreno en que están desplantadas. La zona de mayor destrucción estaba ubicada en su totalidad en terrenos compresibles y abarcó un área de 43 km^2, de un total de 1110 km^2 de área metropolitana en esa época.

EVALUACION DE EDIFICIOS DAÑADOS

Todo edificio dañado que quede en pie después de un temblor intenso puede ser reparado, pero esto debe decidirse en función de la extensión de los daños, la posible inclusión de una estructura adicional, la confianza del propietario en el ingeniero estructurista y el costo de la reparación comparado con el costo de reposición del edificio, incluyendo la demolición.

Antes de hacer el proyecto de reparación es necesario llevar a cabo una evaluación detallada para establecer la importancia de los daños y sobre todo, la causa de éstos; para ello es necesario un conocimiento sólido acerca del comportamiento de los materiales y de los sistemas estructurales. Es necesario recopilar la mayor información posible sobre el edificio, tal como:

- Estudios del subsuelo
- Planos arquitectónicos y estructurales
- Memoria de cálculo de la estructura
- Características de los materiales empleados. Pruebas de control de calidad
- Bitácora de obra
- Cambios de ocupación o modificaciones realizadas durante la construcción o posteriormente
- Efectos de sismos anteriores. Información sobre posibles reparaciones que se le hayan hecho
- Control periódico de nivelaciones y desplomes (especialmente para edificios construidos sobre suelos blandos).
- Medición de periodos de vibración
- Otra información que se tenga.

Es común que no se disponga de toda la información o que aún existiendo no coincida con la realidad, por lo que es necesario verificarla en el lugar haciendo levantamientos de campo; en muchas ocasiones los acabados arquitectónicos: plafones, revestimientos, etc., cubren los elementos estructurales dificultando su observación directa y habrá que retirarlos parcialmente.

Para conocer las características de los materiales empleados es recomendable usar métodos no destructivos; por ejemplo, en estructuras de concreto reforzado usar esclerómetros o aparatos similares para definir la resistencia y la uniformidad del concreto (complementado con el ensaye de algunos corazones de concreto extraídos de zonas donde no se debilite a la estructura) y el uso de profómetros, que permiten conocer la ubicación, recubrimiento y diámetro del refuerzo sin tener que hacer calas. En ocasiones al hacer calas a una estructura para conocer su refuerzo se debilita notablemente y si al término de la evaluación se concluye que no necesitaba reforzarse, de todos modos es necesario repararla por los daños causados al hacer las calas.

Cabe aclarar que en todo edificio se tienen elementos estructurales como columnas, muros, trabes, y losas que serán los que absorberán los efectos de las cargas de peso propio y acabados, así como las cargas vivas y accidentales, pero además existen los llamados elementos "no estructurales" que como el nombre indica no tienen asignada ninguna función de tipo estructural para colaborar en la rigidez y resistencia de la construcción; entre los elementos no estructurales están los plafones, las ventanas y las instalaciones eléctricas, sanitarias o de otro tipo, así como las fachadas y los muros de colindancia o interiores, cuya función no sea estructural por falta de continuidad en elevación y por su posición asimétrica en planta.

La construcción inadecuada de aquellos elementos no estructurales que por sus caracteríscas intrínsecas pueden colaborar de manera importante a la rigidez de la estructura impidiendo el movimiento de ésta cuando tiembla, ha sido causa de numerosas fallas, sobre todo en aquellos casos en que la resistencia a fuerzas laterales depende solo de columnas y trabes o losas, sin la colaboración de muros estructurales.

Los elementos no estructurales que causan más problemas en este sentido son los muros de mampostería de colindancias o interiores, así como los pretiles en fachadas. Estos elementos tienen una gran rigidez en su plano, en ocasiones mayor que la de la estructura y suelen construirse ligados perfectamente a ésta, sin holguras que permitan su movimiento al ocurrir un sismo. Es obvio que los cálculos hechos sin tomar en cuenta esos elementos no serán representativos de la realidad, y que lo que ocurra durante un sismo intenso en esas condiciones es impredecible

Durante los sismos de septiembre de 1985 pudieron observarse muchos casos en los que la falla del edificio se debió a la construcción inadecuada de los elementos no estructurales, especialmente en aquellos casos en que la ubicación de esos elementos provocó problemas importantes de torsión en planta, o falla frágil de columnas por fuerza cortante debido a su mayor rigidez al estar atiesadas parcialmente por algún pretil de fachada o muro bajo, ref. 1

De acuerdo con la ref. 3, 42% de los edificios gravemente afectados por estos temblores estaban ubicados en esquina; esto se explica por las grandes torsiones en planta generadas por la colaboración, completamente asimétrica, de los muros de colindancia ligados a la estructura, lo que hubiera podido evitarse con una construcción adecuada de esos elementos no estructurales.

También, en varios casos, los daños se hubieran podido evitar o reducir si se hubiera hecho una evaluación adecuada de los efectos de temblores previos menos intensos, como por ejemplo el ocurrido en la Ciudad de México el 14 de marzo de 1979, que produjo muchos daños en elementos no estructurales, que se resanaron sin estudiar las causas, lo que habría revelado problemas asociados a la construcción inadecuada de dichos elementos no estructurales que deberían corregirse, ref. 4.

Desde luego hay que reconocer que en algunos casos el colapso total del edificio fue evitado por la colaboración de muros no estructurales colocados simétricamente en planta y sin discontinuidades en elevación.

Cuando la construcción dispone de muros estructurales bien ubicados en planta y suficientemente rígidos y resistentes para absorber una buena parte de los efectos de fuerzas laterales debidas a sismo, la contribución de elementos no estructurales se minimiza.

Entre otros aspectos importantes que hay que considerar al hacer la evaluación de edificios existentes, además del mencionado en relación con los elementos no estructurales, se encuentran la distribución de cargas, la regularidad de la planta arquitectónica así como de la elevación correspondiente y la disposición y tipo de elementos estructurales utilizados; Arnold y Reitherman, ref. 5, engloban todo lo anterior en el concepto de configuración del edificio y analizan las diferencias en la respuesta de las estructuras a los sismos en función de esto.

Un parámetro muy importante en la evaluación de edificios existentes, sobre todo de los desplantados en suelos blandos, es su periodo de oscilación, el cual se puede medir empleando aparatos especiales, que captan el movimiento del edificio provocando por viento y tránsito de vehículos en su vecindad; también puede provocarse mayor excitación empleando vibradores especiales, lo que resulta costoso, o jalando al edificio con cables de acero y liberándolo. La comparación de los periodos de vibración del edificio con los periodos dominantes en la vibración del suelo permitirá detectar condiciones cercanas a la resonancia, que provocan grandes amplificaciones en la respuesta del edificio a un sismo.

REPARACION DE EDIFICIOS DAÑADOS

Se han empleado distintos métodos para reparar estructuras o incrementar su resistencia y rigidez, refs. 6 y 7. Los más usuales son:

- Encamisado de columnas y/o vigas
- Adición de muros de rigidez de concreto reforzado
- Adición de diagonales de contraventeo metálicas, en varias crujias
- Refuerzo de muros de mamposteria con aplanados de concreto reforzados con malla electrosoldada

La eficiencia de estas soluciones es variable y debe decidirse cual usar en cada caso particular dependiendo de las circunstancias que se tengan.

En ocasiones, para no reparar o reducir al mínimo las necesidades de reparación, se han demolido uno o varios niveles de la construcción, confiando en que al bajar el peso bajará la respuesta de la estructura y la resistencia original será suficiente para evitar nuevos daños.

En mi opinión se debe dar atención especial a las características dinámicas de la estructura resultante, ya que en muchos casos los daños pueden atribuirse a condiciones cercanas a la resonancia en la estructura original, como se discutió anteriormente y si no se cuida este aspecto la estructura modificada puede quedar en peligro de sufrir daños mas severos en movimientos futuros. Por ello es importante, antes de iniciar el proceso de reparación, el tener una explicación razonable de las causas de las fallas.

Con la adición de muros de rigidez o diagonales de contraventeo o con la eliminación de varios niveles, es posible modificar significativamente las características dinámicas de las estructuras para alejarlas de situaciones de respuesta peligrosas, aunque debe cuidarse que al modificarla no se lleve a la estructura a una zona del espectro en que sea más vulnerable, de acuerdo con lo mencionado en la introducción, lo cual dependerá del periodo original de la estructura comparado con el dominante en la vibración del suelo. En algunas ocasiones tal vez la mejor opción sea suministrar amortiguamiento adicional a la estructura (ref. 8), para evitar grandes amplificaciones de la respuesta.

Con el encamisado en columnas únicamente o con el encamisado de trabes y columnas no se logra modificar las características dinámicas de manera apreciable, por lo que pueden quedar en condiciones peligrosas en caso de un nuevo sismo intenso.

La adición de muros o diagonales de contraventeo modifica también de manera importante la forma en que se transmiten a la cimentación los efectos de las cargas de sismo, requiriéndose refuerzo local que es complicado de realizar. Por ésto es conveniente que se empleen muros o tableros contraventeados de la mayor longitud posible, o amortiguamiento adicional, para reducir las concentraciones de carga en cimentación. También se modifica en este caso la distribución de fuerzas sísmicas que deben transmitirse a través de los sistemas de piso a los distintos elementos resistentes, por lo que debe verificarse que esos diafragmas horizontales tienen la rigidez y resistencia suficiente para transmitir las fuerzas a los nuevos elementos, y si no es así deberán reforzarse.

Debe reconocer que la determinación de las características dinámicas de las estructuras no es siempre confiable, por la dificultad de modelar adecuadamente al edificio original dañado más los elementos de refuerzo. En suelos blandos el efecto de interacción suelo-estructura puede aumentar apreciablemente el valor del periodo de vibración calculado con la hipótesis de base rígida (entre más rígida sea la estructura mayor será el efecto de interacción). Conviene verificar experimentalmente las modificaciones que sufran las características dinámicas de la estructura midiendo los periodos antes y después de la rigidización o reparación, con aparatos especiales. Esto permitirá también verificar si el modelo matemático fue satisfactorio.

Los conceptos anteriores fueron plenamente comprobados durante los sismos de septiembre de 1985 en la Ciudad de México en dos edificios que fueron reparados empleando diagonales de contraventeo metálicas por haber sido dañados por el sismo del 14 de marzo de 1979 (ref. 4). Más adelante se hacen comentarios sobre estos edificios.

EJEMPLOS DE APLICACION

a) Centrales telefónicas.- La Compañía Teléfonos de México fue de las más afectadas por el sismo de 1985 pues varios de sus edificios sufrieron colapso total, parcial o daños graves, con la suspensión del servicio durante varias semanas, por lo que decidieron emprender una campaña de reestructuración y refuerzo de todas sus centrales telefónicas ubicadas en zonas sísmicas del país aún cuando no tuvieran daños. Bajo mi dirección se han elaborado más de treinta proyectos de refuerzo de centrales telefónicas de distintos tipos y número de niveles. Una de la características principales de estos proyectos era la necesidad de que las actividades normales de la central no se interrumpieran durante la ejecución de los trabajos, por lo que en general los refuerzos se colocaron en la periferia de los edificios Siempre que era posible se colocaban por el exterior, para no afectar el servicio. Prácticamente en ninguna de las centrales se encontraron problemas asociados a condiciones cercanas a la resonancia, pues en general, a pesar de que las estructuras eran relativamente flexibles por tener entrepisos muy altos, de más de 5 m, el número de niveles no llegó a causar problemas por cercanía de periodos de las estructuras comparados con los del suelo en que estaban desplantadas. En general el problema era de resistencia, sobre todo en el suelo blando de la Ciudad de

México, pues los coeficientes sísmicos se incrementaron a valores cuando menos del orden de 2.4 veces los del diseño original, ya que el coeficiente sísmico base subió de 0.24 a 0.40, pero además, para estructuras del grupo A, el factor de incremento cambió a 1.3 a 1.5, y además, en casi todos los casos las estructuras no satisfacen las nuevas condiciones de regularidad establecidas en el nuevo Reglamento, lo que implica un incremento adicional de 25% en las fuerzas de diseño. Además, en muchos casos tampoco se satisfacen los requisitos para usar factores de comportamiento sísmico de 4 ó 3, ya que los nuevos requisitos para marcos dúctiles son muy severos, por lo que el factor de comportamiento que debe usarse en la mayoría de los casos es 2, lo que puede incrementar un 100% adicional las fuerzas de diseño originales; si se diseñaron con factores de comportamiento mayores que 2.

Para subsanar la falta de resistencia se emplearon marcos de acero con diagonales de contraventeo robustas, adosados a la estructura original o aplanados de concreto de 5 a 10 cm de espesor reforzados con malla electrosoldada a lo largo de los muros de fachada, haciendo trabajar estructuralmente a estos muros que en el proyecto original eran no estructurales.

La figura 2 muestra una planta y elevación típica, indicando donde se colocaron marcos contraventeados adicionales. En las figs. 3 a 5 se muestran detalles de recimentación para recibir los marcos y distintos aspectos de la colocación de éstos.

b) Otro caso en que los daños no pueden atribuirse a las condiciones dinámicas, es el de un edificio de hospital localizado en una zona de máximos daños, con varios colapsos totales en un radio de 100 m. El edificio tiene ocho pisos y sótano (figs. 6 y 7). La estructura original era a base de marcos de concreto reforzado y muros de concreto en algunas crujías por lo que era relativamente rígido comparado con las propiedades de suelo; sin embargo, hubo fallas importantes en las columnas del sótano causadas por golpes contra la losa del piso de un estacionamiento anexo cuyo nivel era aproximadamente 50 cm abajo del nivel de piso dentro del edificio; la cimentación tiene pilotes de control, fue necesario cambiar la mayoría de los puentes que transmiten la carga a los pilotes, pues se dañaron a causa del golpeteo. En el resto del edificio los daños fueron menores, con algunas losas de piso agrietadas, lo que se atribuyó a efectos de fuerza cortante

Las normas de emergencia publicadas el 18 de octubre de 1985, que se emplearon para la reparación, pedían la reestructuración de cualquier edificio que hubiera sufrido daños mayores. El nuevo coeficiente sísmico era 67% mayor que el del reglamento en vigor antes del sismo, por lo que, para lograr que la estructura cumpliera con las nuevas normas fue necesario reforzar también los niveles superiores del edificio. Las columnas del sótano fueron encamisadas y algunos tramos del muro de retención, que no llegaban hasta el nivel de planta baja para dar iluminación al sótano, fueron cerrados para tener mayor capacidad resistente. Se agregaron también varios muros de concreto reforzado en los pisos superiores para satisfacer las demandas de resistencia de las normas de emergencia, estos muros se anclaron a los marcos empleando tramos de varillas que se alojaban en taladros y se rellenaban con resinas epoxy.

Las grietas de las losas de niveles superiores fueron rellenadas con resinas epoxy, y para aumentar su resistencia al corte se aplicó presfuerzo a las losas en la dirección longitudinal del edificio (fig. 8). No fue necesario incrementar el número de pilotes de la cimentación. La losa de estacionamiento que causó el problema se demolió a lo largo de su contacto con el edificio, dejando una trinchera para absorber movimientos futuros.

Se midieron los periodos de vibración en condiciones ambientales después de la reparación, obteniendo valores de 0.62 seg. en dirección transversal y 0.53 seg. en la dirección longitudinal Estos valores concuerdan razonablemente bien con los calculados. Se considera que estos periodos garantizan que el edificio tendrá un respuesta baja en movimientos futuros ya que están suficientemente lejos de los periodos dominantes del movimiento del suelo en ese lugar; asimismo la causa principal del daño (la losa del estacionamiento) fue eliminada.

c) Las figuras 9 a 14 ilustran los refuerzos a base de marcos contraventeo adicionales en dos edificios cuyas condiciones dinámicas los hacían muy vulnerables a los efectos sísmicos, aunque su periodo era mayor que el dominante en el suelo. La rigidez de los marcos adicionales permitió que se lograra una reducción importante de los periodos, para que la respuesta sea adecuada.

COMENTARIOS ADICIONALES

Pruebas en edificios reparados

El excelente comportamiento de edificios rigidizados con diagonales de contraventeo durante el sismo de septiembre de 1985 en la Ciudad de México, pudo apreciarse en tres edificios de mediana altura que fueron reparados empleando este sistema tras haber sido dañados por temblores anteriores a 1985, (refs. 4 y 9), y que se comportaron muy satisfactoriamente en esta ocasión. Con el patrocinio de la Fundación Nacional de Ciencias de los Estados Unidos, dos de estos edificios fueron estudiados dentro de un proyecto de investigación encabezado por Douglas Foutch de la Universidad de Illinois y yo mismo. En enero de 1987 llevamos a cabo pruebas de vibración forzada en ambos edificios empleando los vibradores de masas excéntricas propiedad del Instituto Tecnológico de California, y varios alumnos realizaron el análisis elástico e inelástico paso a paso de distintos modelos matemáticos de los edificios. La interpretación de los resultados de las pruebas de vibración forzada y su reconciliación con los de los modelos matemáticos se resume en las figuras 15 y 16, (ref. 12).

Como puede verse en dichas figuras, una parte importante de los desplazamientos laterales se debe a la rotación de la base, debido a las características del subsuelo blando de la Ciudad de México, Esto disminuyó un poco la eficiencia del sistema de contraventeo, pues por ejemplo, en el edificio de Durango el periodo con base empotrada sería de 0.90 seg. mientras que el periodo con interacción es de 1.26 seg.; sin embargo, la solución funcionó bastante bien como se indicó anteriormente, ya que cambió radicalmente el periodo que tenía la estructura antes de la rigidización, de 1.86 seg., sacándola de una condición cercana a la resonancia, que fue la que le provocó los daños en un temblor mucho menos intenso que el de 1985. Para el edificio de Parque España la situación es similar. Se puede afirmar que si no se hubieran rigidizado o reforzado estos edificios hubieran colapsado en 1985.

Adición de dispositivos para absorber energía

Una innovación en ingeniería sísmica es la adición de amortiguamiento externo a las estructuras para absorber energía y reducir su respuesta. Se ha demostrado en numerosos artículos técnicos los efectos benéficos del amortiguamiento para reducir la amplificación de los movimientos del terreno; la ventaja fundamental de estos nuevos sistemas es que no es necesaria la formación de articulaciones plásticas para que se reduzca la respuesta, ya que los amortiguadores pueden diseñarse de tal manera que empiecen a actuar antes que la estructura llegue al comportamiento inelástico.

Se han propuesto varios tipos de amortiguador diferentes, (refs. 8, 10 y 11), uno emplea capas de material viscoelástico en contacto con placas de acero para absorber la energía por deformación; el otro usa balatas de freno para disipar energía por fricción contra placas de acero, otro más emplea placas que fluyen plásticamente. Todos pueden introducirse en crujías contraventeadas.

La Universidad de California en Berkeley ensayó un modelo de edificio de acero de nueve niveles equipado con distintos tipos de disipadores en la mesa vibradora instalada en Richmond, con excelentes resultados.

También Hanson, en la Universidad de Michigan, ha estudiado los efectos de amortiguamiento suplementario en los edificios (ref. 8), con resultados muy promisorios. Los dispositivos de Hanson han sido empleados en la reestructuración de tres edificios de la Ciudad de México, (ref. 12).

CONCLUSIONES Y RECOMENDACIONES

La reestructuración de los edificios puede ser exitosa si se toman en cuenta una serie de detalles; debe verificarse que las propiedades dinámicas de la estructura resultante sean adecuadas con respecto a las características del movimiento del terreno; el flujo de fuerzas entre la estructura original y los refuerzos debe estudiarse cuidadosamente para conectarlos correctamente. Con frecuencia es necesario reforzar las columnas que enmarcan las crujías contraventeadas debido a las fuerzas adicionales que atraen, en virtud de la redistribución de efectos sísmicos causados por la mayor rigidez de estas crujías. Es necesario verificar también que la cimentación tenga la resistencia y rigidez suficientes para absorber adecuadamente la nueva distribución de fuerzas. También es muy importante asegurarse de que los diafragmas horizontales tengan la rigidez y resistencia suficientes para transmitir las fuerzas a los nuevos elementos; si no es así, es necesario reforzarlos.

El empleo de amortiguadores para reducir la respuesta durante temblores intensos constituye una técnica muy promisoria que ya se ha empleado en la reestructuración de edificios dañados y está en proceso en la construcción de edificios nuevos.

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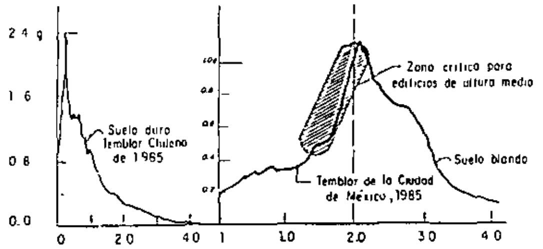
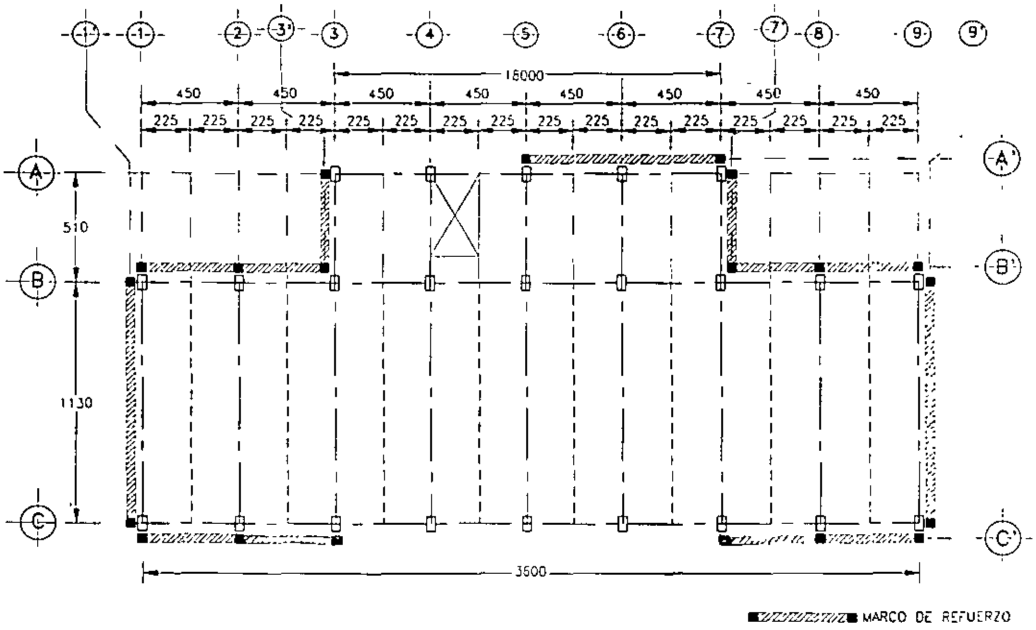


Fig 1 Espectros de respuesta tipicos para terrenos duros y blandos, para 5 % de amortiguamiento y comportamiento elastico



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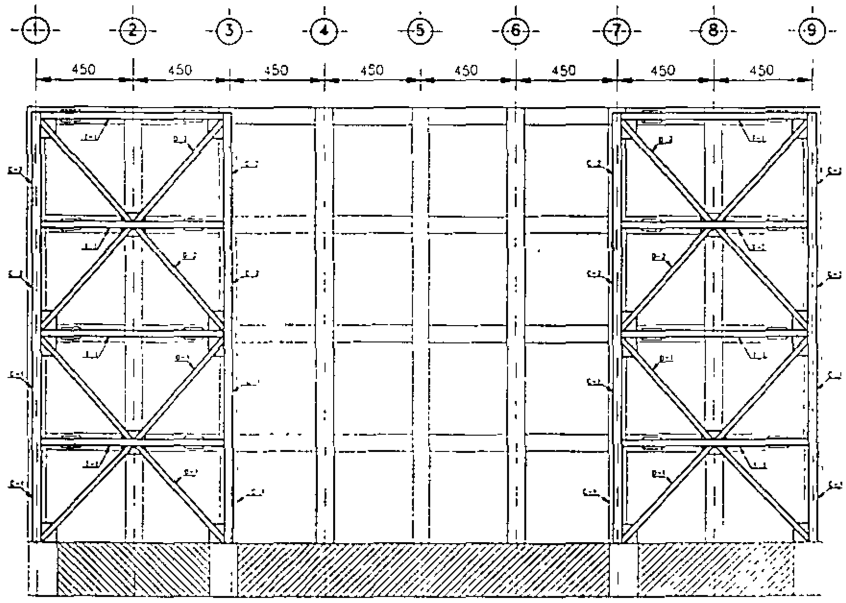


ELEVACION (MARCO EJE C')

FIG 2

Fig 3

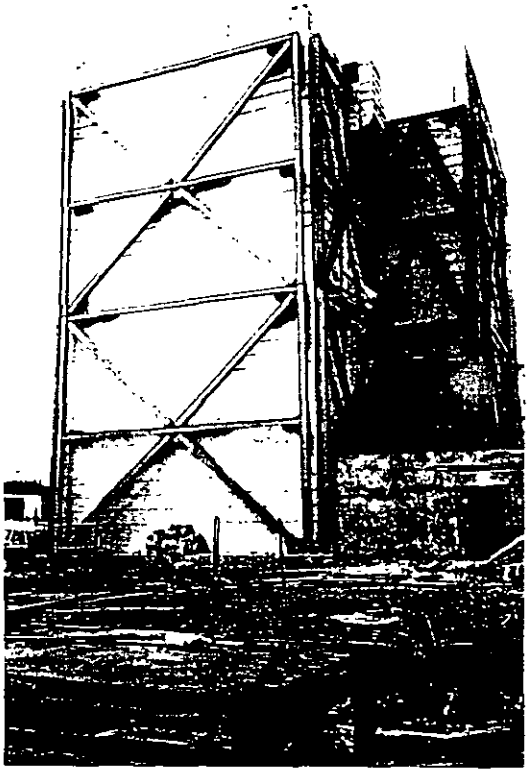
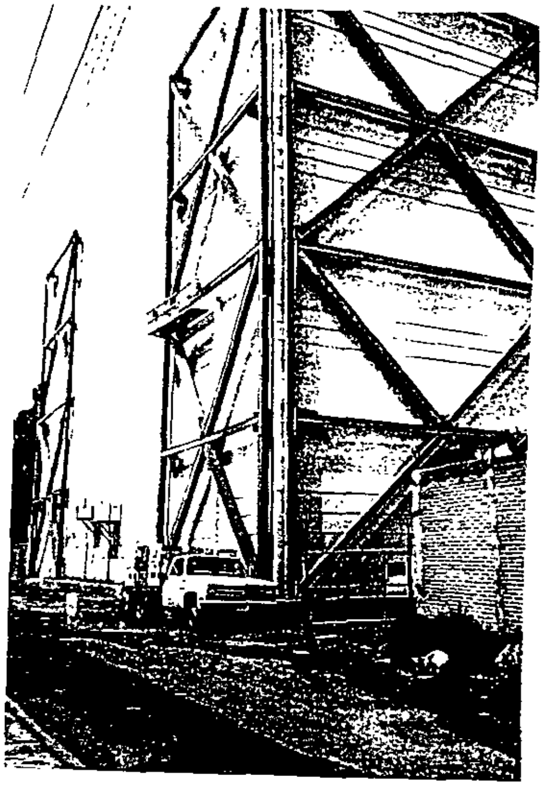


Fig 4



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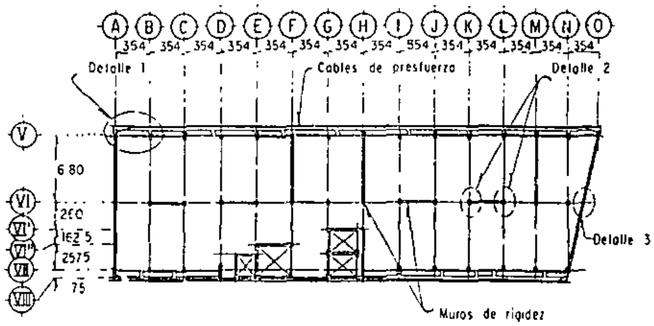


Fig 6 Planta tipo Hospital

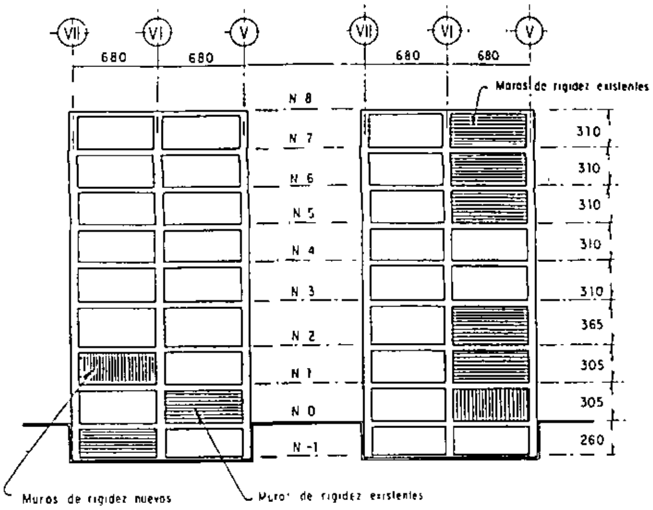


Fig 7. Elevación de marcos J y N . Hospital

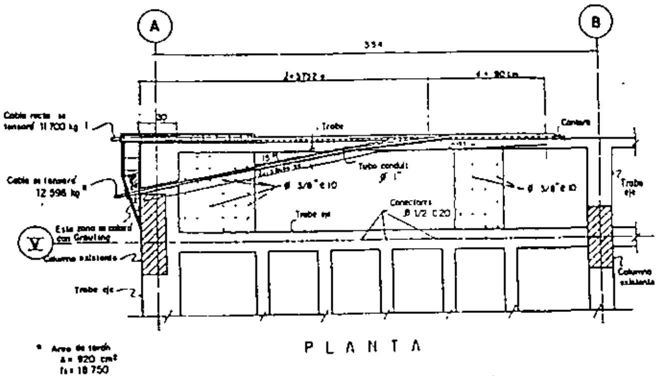


Fig 8 Detalle Nº 1 (Ver fig 6) Hospital

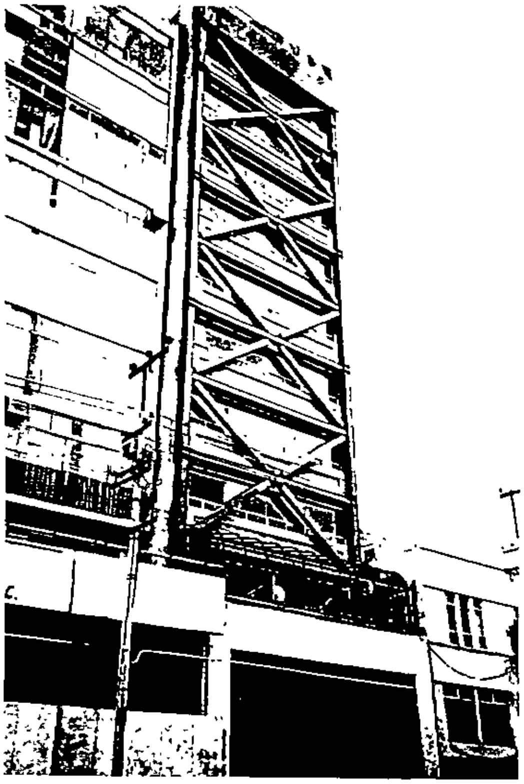


Fig 9

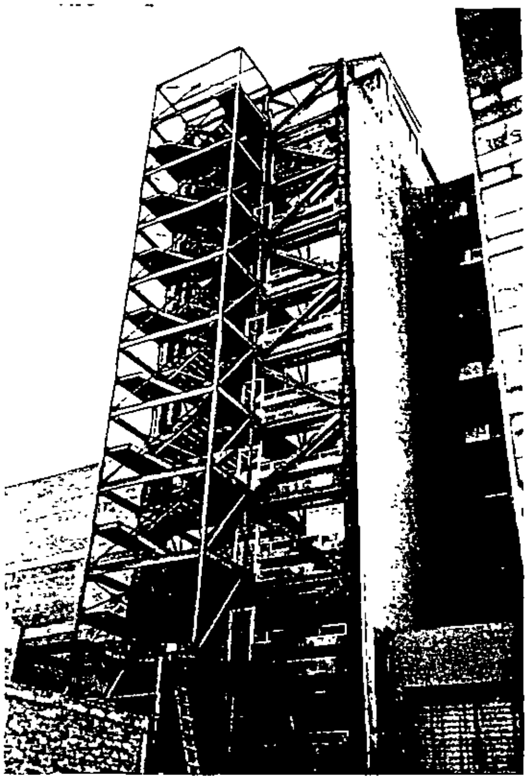
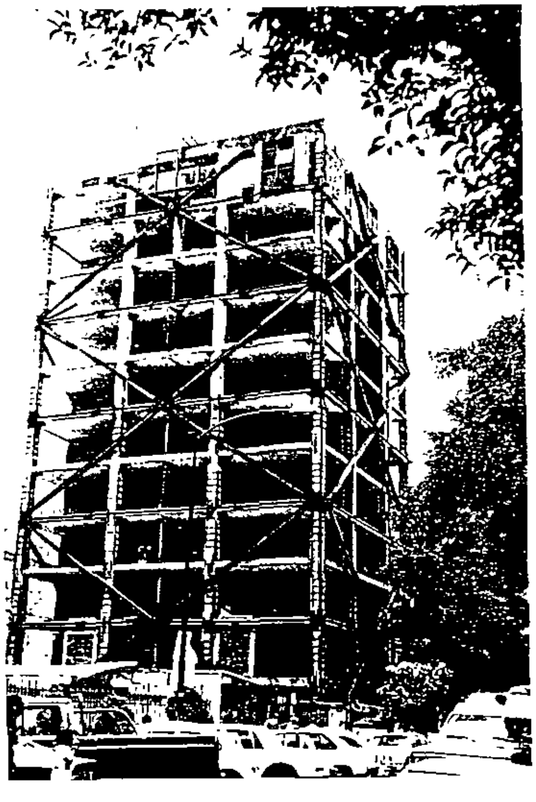


Fig 10

Fig 11



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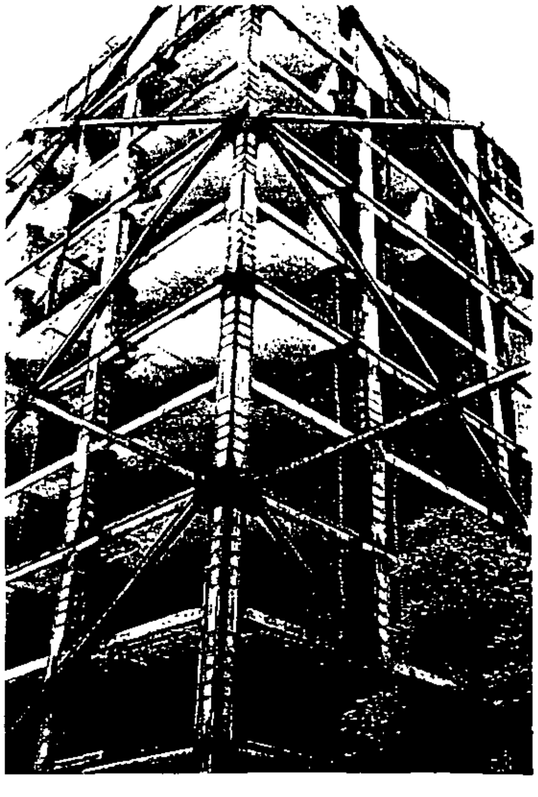


Fig 13

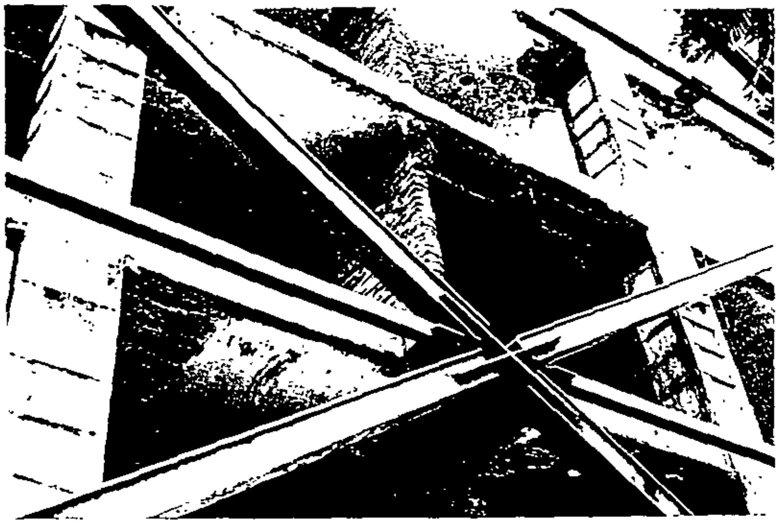
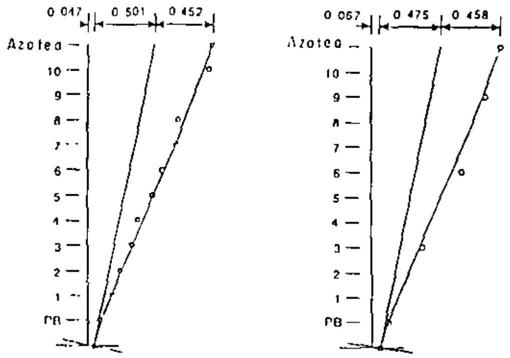


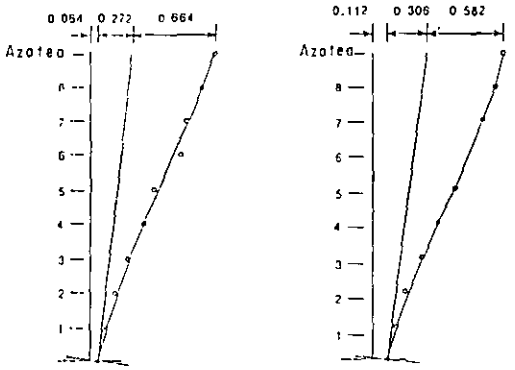
Fig 14.



Forma del primer modo E - W
Periodo : 1.26 seg
Amort. : 2.8 %

Forma del primer modo N - S
Período : 0.99 seg.
Amort . 4.8 %

Fig 15 Formas modales para el Edificio en Calle Durango



Forma del primer modo N-S
Período . 1.11 seg
Amort · 3 0 %

Forma del primer modo E-W
Período · 0 90 seg.
Amort . 6.9 %

Fig 16 Formas modales para el Edificio en Parque España

FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

XXVI CURSO INTERNACIONAL DE INGENIERIA SÍSMICA

MOUDLO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

REFORZAMIENTO Y OBSERVACIONES POSTERIORES AL SISMO DE 1985, DE UN EDIFICIO EN LA CIUDAD DE MEXICO

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PALACIO DE MINERIA
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REFORZAMIENTO Y OBSERVACIONES POSTERIORES AL SISMO DE 1985, DE UN EDIFICIO EN LA CIUDAD DE MEXICO.

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1.- RESUMEN.

Se describen los daños causados en un edificio de la ciudad de México por el sismo del 14 de marzo de 1979, los criterios empleados para la reparación de los daños, el comportamiento del edificio ante el sismo del 19 de septiembre de 1985 y los estudios que se hicieron en él después de dicho temblor para explicar su buen comportamiento

2.- INTRODUCCIÓN.

El sismo del 14 de marzo de 1979, de magnitud 7.6 y con epicentro en las costas de Guerrero, fue el más intenso sentido en la Ciudad de México después del ocurrido el 28 de julio de 1957, que fue el que sirvió de base para los reglamentos de diseño por sismo de la ciudad de México El sismo de 1979 afectó un número importante de edificios, provocando varios colapsos totales Entre los edificios afectados se encontraba un condominio de consultorios médicos ubicado en la Colonia Roma, el cual se analiza en este trabajo, ver ref. 1.

3.- DESCRIPCIÓN DEL EDIFICIO.

El edificio tiene 12 niveles y caseta de elevadores La estructura es de concreto reforzado, su planta mide 11 9 m por 21 3 m , con una altura de 36.4 m sobre el nivel de desplante, excluyendo la caseta de elevadores y escaleras La figura 1 muestra la planta típica.

Las fuerzas laterales en la estructura original eran resistidas por los marcos 1 a 5 en la dirección este – oeste (transversal) y por los marcos A y C en dirección norte – sur (longitudinal) Los marcos 1 y 5 tienen sólo dos columnas cada uno, figura 2, y son idénticos salvo por que la trabe de planta baja se omitió en el marco 1 para permitir el acceso de vehículos al sótano. En ambos marcos se emplearon trabes peraltadas (1 35 m) como parte de las fachadas Estas trabes acortan la longitud efectiva de las columnas de estos marcos y los hacen mucho más rígidos que los marcos 2 a 4, así como más rígidos de lo supuesto en el análisis original, ya que cuando el edificio fue diseñado no se empleaban aún computadoras para análisis estructural, y éstas se analizaban con métodos aproximados que en general no tomaban en cuenta la rigidez adicional producida por la intersección de trabes peraltadas y columnas (efecto de nudo) subestimando por tanto la rigidez de entrepiso, lo que propiciaba la posible falla de las columnas al absorber esos marcos fuerzas mayores a las calculadas En la figura 2 se muestran los daños causados en 1979.

Los marcos 2 a 4 son idénticos, con 3 columna trabes acarteladas como muestra la figura 3. El sistema de piso esta constituido por una losa de 5 cm de espesor reforzada con nervaduras en la dirección norte – sur a cada 60 cm

Los marcos A y C son idénticos, el marco B no está conectado con la losa por lo que no forma parte del sistema resistente a fuerzas laterales. Los marcos A y C tienen muros de colindancia no estructurales, de mamposteria de tabique rojo recocido, desligados de los marcos por medio de una holgura rellena con celotex a los lados y extremo superior en cada crujía. Se consideró que estos muros no colaboraban con los marcos para resistir las fuerzas sísmicas.

La cimentación consiste en un cajón de concreto reforzado, con muros de contención de 20 cm de espesor y con trabes de liga en los ejes de columnas, con un peralte total de 2.40 m. soportado por pilotes de fricción. Los resultados de los estudios que se hicieron al edificio indicaron que esta cimentación se comporta como cuerpo rígido. En un estudio geotécnico realizado como parte los estudios se encontró que el edificio está sobre estra de arcilla muy blanda de 40 m de espesor, con contenidos de humedad promedio de 300%, ref. 2.

4.- DAÑOS OBSERVADOS DESPUES DEL SISMO DE 1979.

El edificio fue seriamente dañado por el sismo de 1979 Las conexiones trabe-columna de los marcos 1 y 5 sufrieron agrietamientos severos, en el marco 5 el acero de refuerzo en el primer nivel quedó expuesto, figura 4. Las columnas de los primeros cuatro niveles mostraban agrietamientos diagonales y verticales, figura 5. Las vigas peraltadas también estaban bastante agrietadas. Los muros interiores tenían agrietamientos importantes. Después de estudiar el problema se concluyó que el daño podría atribuirse a la subestimación de la rigidez de los marcos antes citada Además, la medición de periodos del edificio reveló que en dirección transversal era relativamente flexible. con periodo de 1 87 seg, que es alto para un edificio de 12 niveles y que estaba relativamente cercano a los periodos dominantes en el terreno en que estaba desplantado el edificio. lo que propiciaba una mayor respuesta. debido a la posible resonancia. El edificio fue evacuado para poder repararlo

5.- CRITERIOS DE REPARACION EMPLEADOS

Existen diversas técnicas para la reparación de edificios dañados, desde inyección de las grietas con resinas epóxicas, encamisado de columnas y trabes, adición de muros de rigidez o refuerzo de los muros existentes o inclusión de elementos diagonales de contraventeo metálicos. conectados a la estructura original, entre otras. ver ref 3

Antes de decidir cual técnica emplear se deben identificar las causas de los daños, pues si no se hace ésto se corre el riesgo de dejar a la estructura en una condición más vulnerable después de la reparación. como puede ocurrir cuando el edificio tiene un periodo mayor al dominante en el terreno en el cual está desplantado y se le rigidiza. lo que puede acercarlo a la condición de resonancia y aumentar la respuesta en temblores futuros.

En el caso del edificio en estudio se concluyó, como se mencionó antes, que los daños podrían atribuirse a la subestimación de la rigidez de los marcos de fachada. lo que motivó que absorbieran fuerzas mayores a las consideradas en el diseño y fallaran. combinado con una mayor respuesta del edificio debido a sus características dinámicas, con periodos cercanos a los dominantes en la vibración del terreno, pero inferiores a éstos.

Para calcular las fuerzas sísmicas a que se vería sometida la estructura se aplicó el Reglamento de construcciones para el Distrito Federal de 1976. ref 4. que especificaba un coeficiente sísmico de 0.24 g. que se podía reducir mediante factores de comportamiento sísmico que variaban entre 1 y 6 Para la reparación se consideró un factor de comportamiento sísmico de 2, que es conservador, pero se juzgó adecuado para tomar en cuenta las dificultades inherentes a los trabajos de reparación

Se estudiaron dos maneras de repararlo, tratando en ambos casos de reducir los efectos sísmicos en los miembros afectados. El primero consistió en la adición de muros de concreto en la zona de los cubos de elevadores y escaleras; sin embargo. esta solución no era lo suficientemente rígida para reducir los efectos en los elementos dañados e introducía momentos flexionantes elevados en las vigas que conectaban con esos muros. por lo que se estudió una segunda opción empleando grandes armaduras verticales de acero, de todo el ancho del edificio, adosadas a los marcos de fachada anterior y posterior, con refuerzos locales en las losas para trasmitir adecuadamente las fuerzas sísmicas que serían absorbidas por estas armaduras, figura 6 Las conexiones entre la estructura original y los marcos de refuerzo se hicieron a base de placas de acero y pernos, conectando en tres puntos de cada nivel a las trabes peraltadas, ver figura 7. Las columnas de los marcos de refuerzo y las diagonales del primer nivel están formadas por cuatro placas soldadas en cajón, las diagonales de los pisos superiores son dos canales unidos por placas. con objeto de tener relaciones de esbeltez relativamente bajas En el primer nivel fue necesario poner solamente dos diagonales para permitir el acceso de vehiculos y personas al edificio, en los niveles superiores se usaron cuatro diagonales en cada nivel de común acuerdo con el arquitecto que proyectó el edificio.

Se colocaron pilotes adicionales metálicos y se prolongó la cimentación para conectarla con los dados de esos pilotes y lograr un trabajo monolítico; dada la gran rigidez de los nuevos marcos con respecto a la estructura original, se diseñaron para resistir la totalidad de las fuerzas sísmicas de la dirección transversal, despreciando la contribución de la estructura original

Las columnas dañadas en los cuatro niveles inferiores fueron reparadas con un encamisado de placas de acero, rellenado con mortero con aditivo estabilizador de volumen, las grietas en las trabes se inyectaron con resinas epóxicas

En la dirección longitudinal se reforzaron por el interior del edificio los muros de colindancia de los marcos A y C entre los ejes 1-2 y 4-5 en toda la altura. Las placas de celotex que llenaban las juntas entre la mampostería y la estructura fueron eliminadas, rellenando el espacio con mortero, se desprendieron los acabados de los muros, se colocaron clavos largos en ambas direcciones de los cuales se sujetó una malla electrosoldada y se colocó un aplanado de concreto de 5cm. de espesor mínimo, ver figura 8.

Una vez terminada la reparación se volvió a hacer una medición de los periodos de vibración del edificio, encontrando que en la dirección transversal se habia reducido 1.87 a 1 15 seg, lo que implicaba un incremento en la rigidez global de 164%, esto es, la nueva rigidez era 2 64 veces la original La medición se hizo en condiciones ambientales, con un sismógrafo portátil, midiendo la excitación del edificio provocada por viento y tránsito de vehiculos en su vecindad El incremento promedio en el peso del edificio por el refuerzo fue de 6 5%, que es relativamente bajo

6.- COMPORTAMIENTO DEL EDIFICIO ANTE LOS SISMOS DE SEPTIEMBRE DE 1985

Es bien conocido que los sismos de 1985 han sido los más destructivos en la historia de la ciudad de México, afectando a más de 5000 inmuebles con alturas comprendidas entre 1 y 24 niveles, pero concentrándose de manera especial las afectaciones en los edificios de altura media, entre 6 y 15 niveles, ref. 5, debido, principalmente, al incremento en la respuesta de estas estructuras por la cercanía de sus periodos de vibración a los periodos dominantes en la vibración del suelo, lo que se puede apreciar claramente en la forma de los espectros de respuesta obtenidos, como el de SCT, figura 9, correspondiente a una zona de la ciudad donde el perio dominante del suelo es de 2 seg. Después de 1985 hicieron una serie de mediciones de los periodos del suelo en distintos sitios de la ciudad, principalmente en la zona de terrenos compresibles, encontrando que varían entre 1 seg. aproximadamente, cerca de la zona de transición al poniente de la ciudad, a más de 4 seg en la zona oriente de la misma, ref. 6.

El edificio en estudio se ubica en una zona donde el periodo del suelo es cercano a los 2 seg. como en SCT, por lo que se puede ver claramente, en la fig. 9, que al haber reducido el periodo de vibración del edificio de 1.87 a 1 15 seg, la respuesta ante el sismo del 19 de septiembre de 1985 se redujo aproximadamente a la tercera parte, lo que explica que el edificio no haya tenido ningún daño durante este sismo , a pesar de que el refuerzo fue calculado para un sismo de diseño mucho menor que el que se presentó

7.- ESTUDIOS HECHOS EN EL EDIFICIO.

El excelente comportamiento del edificio atrajo el interés de investigadores internacionales. Con el patrocin de la Fundación Nacional de Ciencias de los Estados Unidos se hizo una investigación muy completa encabezada por el Dr Douglas Foutch , de la Universidad de Illinois y por el autor de este trabajo que a la sazón era Investigador Nacional en la División de Estudios de Posgrado de la Facultad de Ingeniería de la UNAM, ref 2. El proyecto incluyó estudios del subsuelo en el sitio, vibración forzada del edificio empleando unos vibradores de masas excéntricas propiedad del Instituto Tecnológico de California, instrumentando al edificio para poder medir las deformaciones provocadas por la excitación y los esfuerzos inducidos en los elementos de refuerzo. Se hicieron estudios teóricos con análisis modal elástico y paso a paso, incluyendo efectos de interacción suelo-estructura, lo que permitió que varios alumnos de la Facultad de Ingeniería de la UNAM y de la Universidad de Illinois pudieran elaborar tesis de Maestría y de Doctorado. Dentro de este proyecto se estudió también otro edificio dañado en 1979 y reforzado con asesoría del suscrito, que también tuvo un comportamiento bastante satisfactorio en 1985, refs 1, 2, 7

Los resultados de las pruebas de vibración forzada confirmaron que la ausencia de daños en el edificio se debió a la adecuación de sus características dinámicas para alejarse de una condición cercana a la resonancia Dos vibradores de masas excéntricas se colocaron en la azotea del edificio, de tal forma que se pudo excitar al edificio en vibración transversal, longitudinal y en torsión acomodando la posición inicial de las masas de los vibradores adecuadamente, figura 10. Se hicieron barridos de frecuencias de excitación en cada caso para identificar varias formas modales. Al aumentar la frecuencia a la que giraban las masas se reducían éstas, con objeto de controlar las fuerzas aplicadas al edificio y no provocar algún daño, los periodos y amortiguamientos obtenidos se resumen en la tabla 1

TABLA 1 Propiedades dinámicas medidas en el edificio

Modo	Periodo (segundos)	Amortiguamiento (% del crítico)
1 EW	1.263	3
2 EW	0.285	5
1 NS	1.005	5
2 NS	0.219	9
1 Torsión	0 486	6

El periodo calculado en dirección transversal (E-W) suponiendo la estructura empotrada en su base es de 0.90 seg. Al considerar efectos de interacción suelo estructura los valores calculados coinciden razonablemente con los periodos medidos en ambas direcciones, sin embargo, como puede verse en la figura 11, las formas modales en ambas direcciones también se ven fuertemente influenciadas por la interacción suelo-estructura, ya que para la dirección transversal aproximadamente el 55% de la deformación del último nivel se debe a translación (4.7%) y a rotación de la base (50%), quedando solo 45% de esa deformación asociado a la deformación de la estructura. Para la dirección longitudinal se obtuvieron resultados similares. Por lo que respecta al amortiguamiento es notable la diferencia en ambas direcciones, pues en la transversal es del orden del 3%, asociado al comportamiento de una estructura metálica, mientras que en la dirección longitudinal se obtuvo cerca del 5%, que se asocia bien a una estructura de concreto reforzado

Se considera que esta influencia de la interacción suelo-estructura es la más grande medida en un edificio, y es el resultado de la combinación de suelo muy blando y edificio rígido Es importante tomar ésto en cuenta pues en este caso 40% de la rigidez del edificio se perdió debido a la flexibilidad en la base.

Los análisis detallados efectuados revelaron que la máxima fuerza cortante en el edificio, en la dirección este-oeste llegó a 0 27W, empleando el acelerograma de SCT. Un cortante en la base de 0.30W hubiera ocasionado que alguno de los elementos metálicos del refuerzo alcanzaran su nivel máximo de esfuerzo admisible de acuerdo con lo especificado por el Instituto Americano de la Construcción en Acero, AISC, en 1980. Esto indica que si el diseño se hubiera hecho con un factor de comportamiento menos conservador y no se hubiera despreciado la resistencia de la estructura original podría haberse tenido algún daño en 1985.

Al analizar la estructura original con el acelerograma obtenido en SCT en 1979, se obtuvo una cortante en la base de 0 09W, ver figura 9, con una resistencia última del edificio del orden de 0.13W. El daño observado en 1979 es consistente con estos valores. La respuesta elástica del edificio original al sismo de 1985 hubiera arrojado un cortante en la base del orden de 0 80W, lo que seguramente lo hubiera llevado al colapso, ver figura 9 y ref. 2.

8.- CONCLUSIONES.

La reestructuración de edificios empleando elementos metálicos de contraventeo puede ser exitosa si se toma en cuenta una serie de detalles, como verificar que las propiedades de la estructura resultante, incluyendo efectos de interacción suelo-estructura, sean adecuadas con respecto a las características del movimiento del terreno; el flujo de fuerzas entre la estructura original y los refuerzos debe estudiarse cuidadosamente para conectarlos adecuadamente. Con frecuencia es necesario reforzar las columnas que enmarcan las crujías contraventeadas debido a las fuerzas adicionales que atraen en virtud de la redistribución de efectos sísmicos causados por la mayor rigidez de estas crujías Es necesario verificar que la cimentación tenga la resistencia y rigidez suficientes para absorber adecuadamente la nueva distribución de fuerzas También es muy importante asegurarse de que los diafragmas horizontales tengan la rigidez y resistencia suficiente para transmitir las fuerzas a los nuevos elementos y si no es así, reforzarlos.

La relación de esbeltez de los elementos de contraventeo debe ser suficientemente baja para eliminar problemas de pandeo Se recomienda un máximo del orden de 100.

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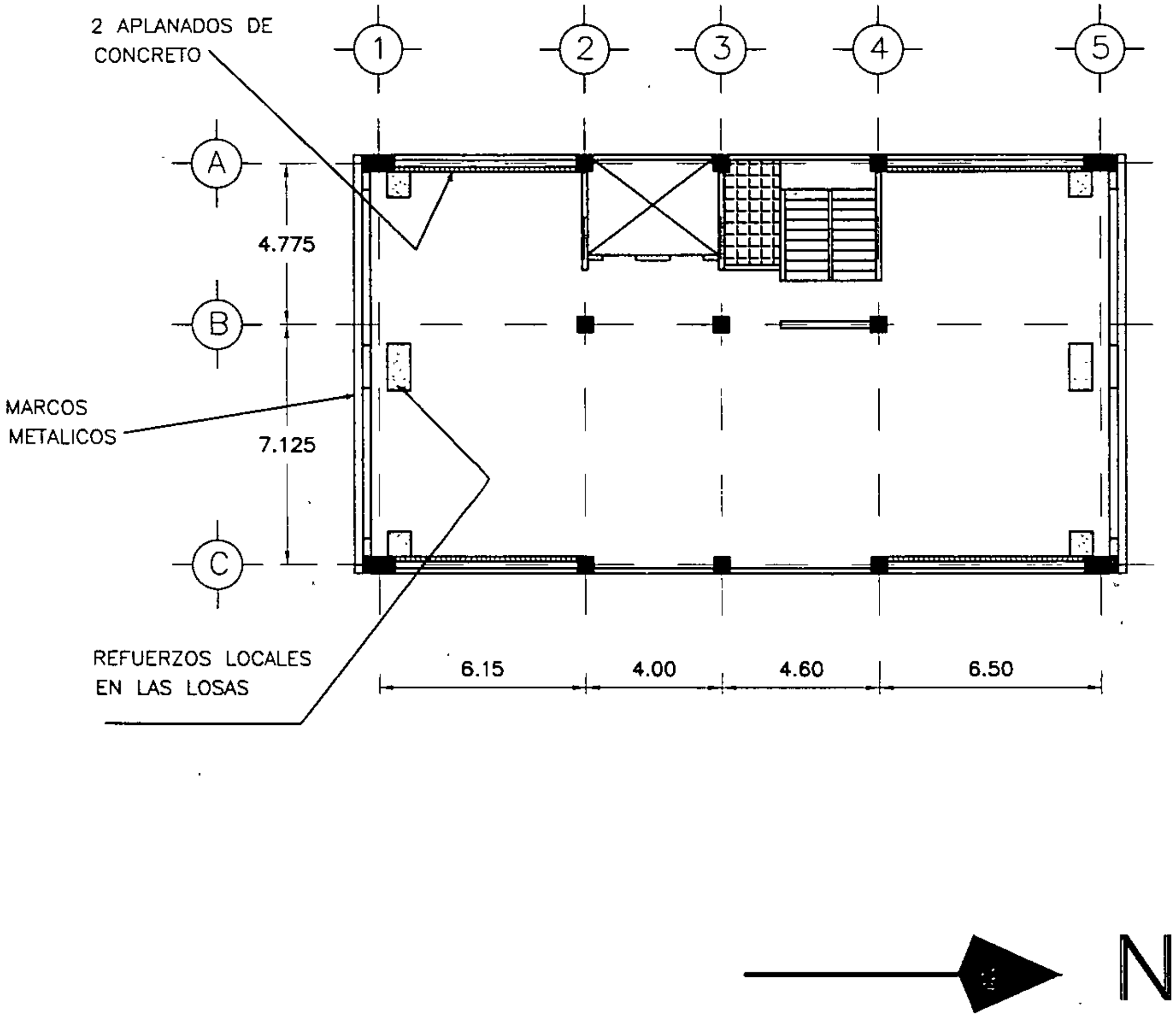


Fig. 1 Planta típica del edificio.

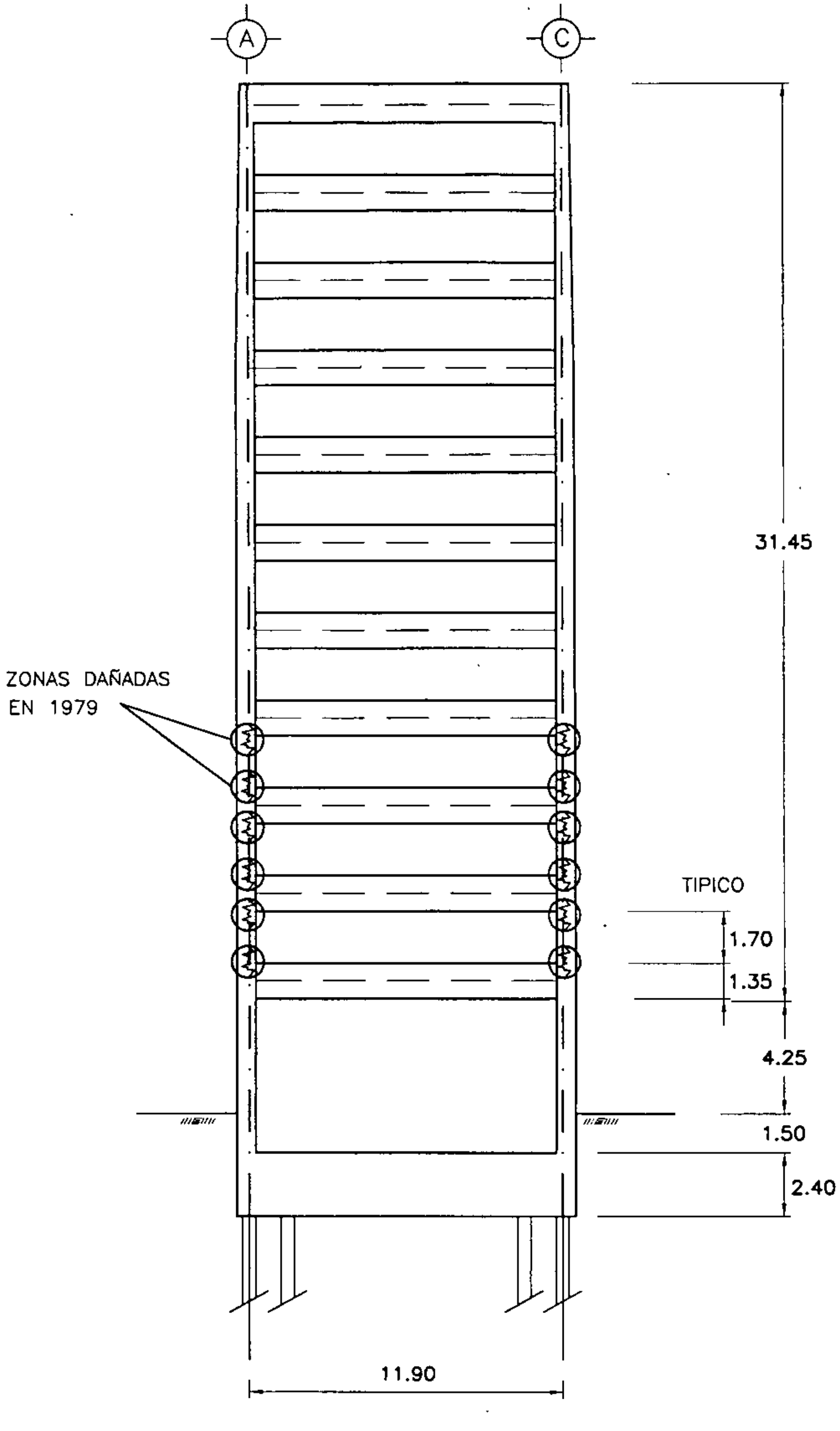


Fig.2 Elevación de marco 1, mostrando daños causados en 1979

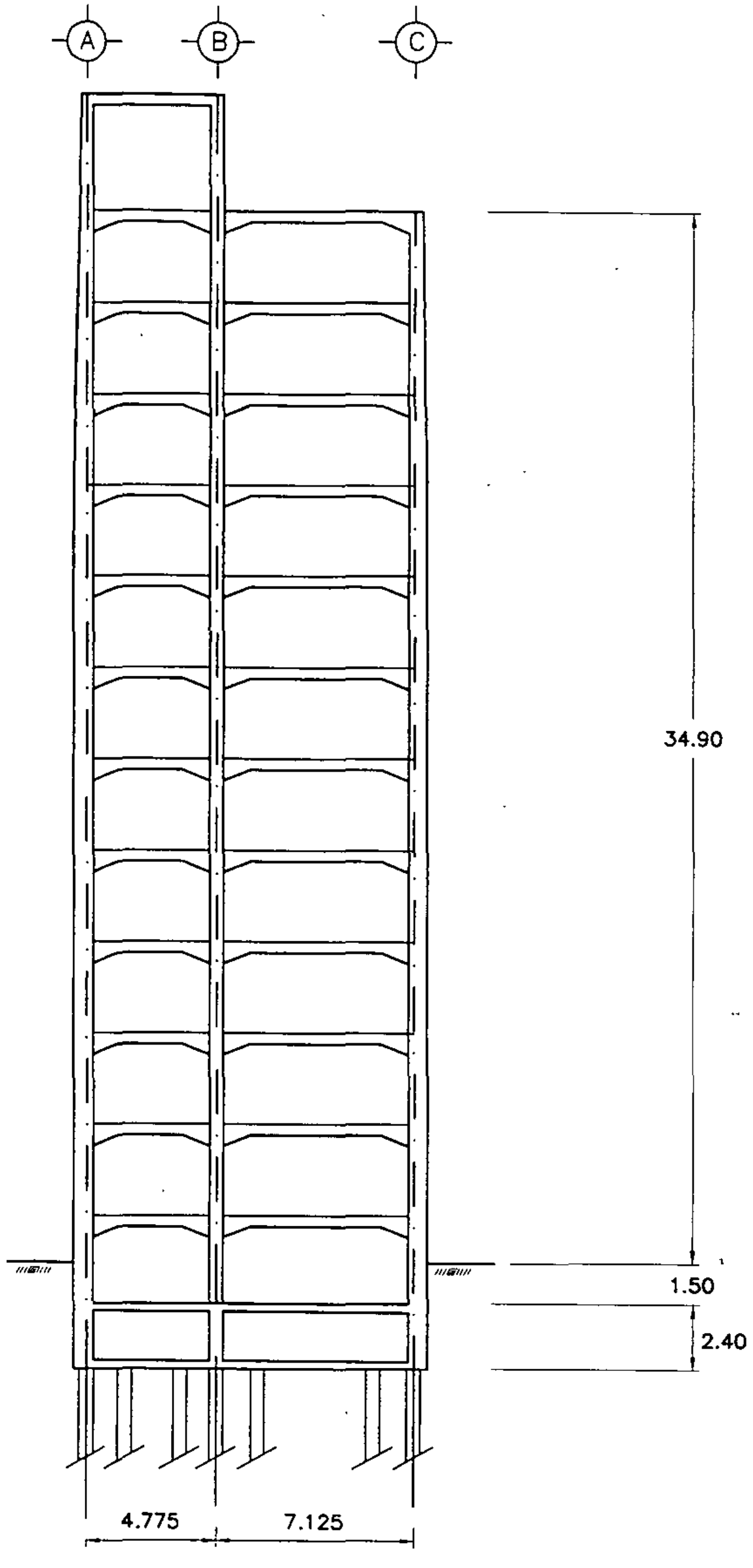


Fig.3 Elevación de marcos 2,3 y 4.

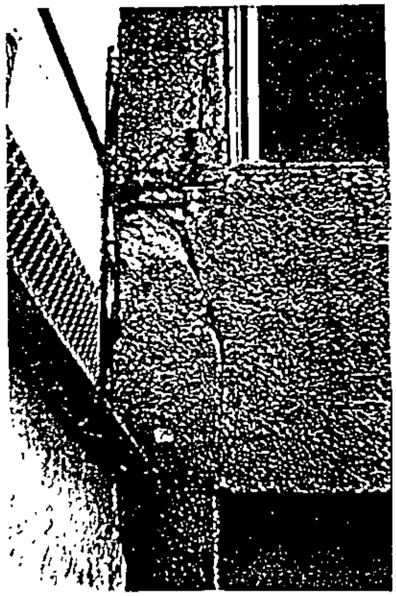


Fig. 4 Daños en el marco 5,
fachada posterior.

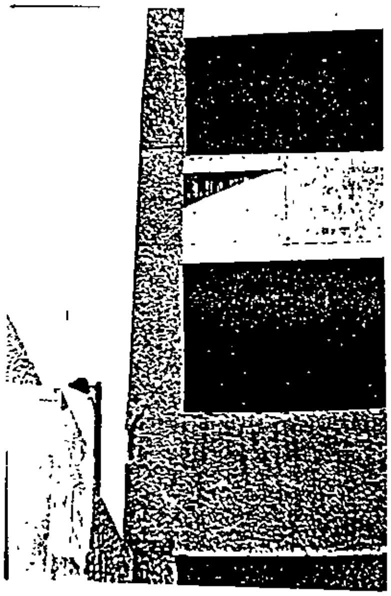


Fig. 5 Daños en el marco 1, fachada a la calle.

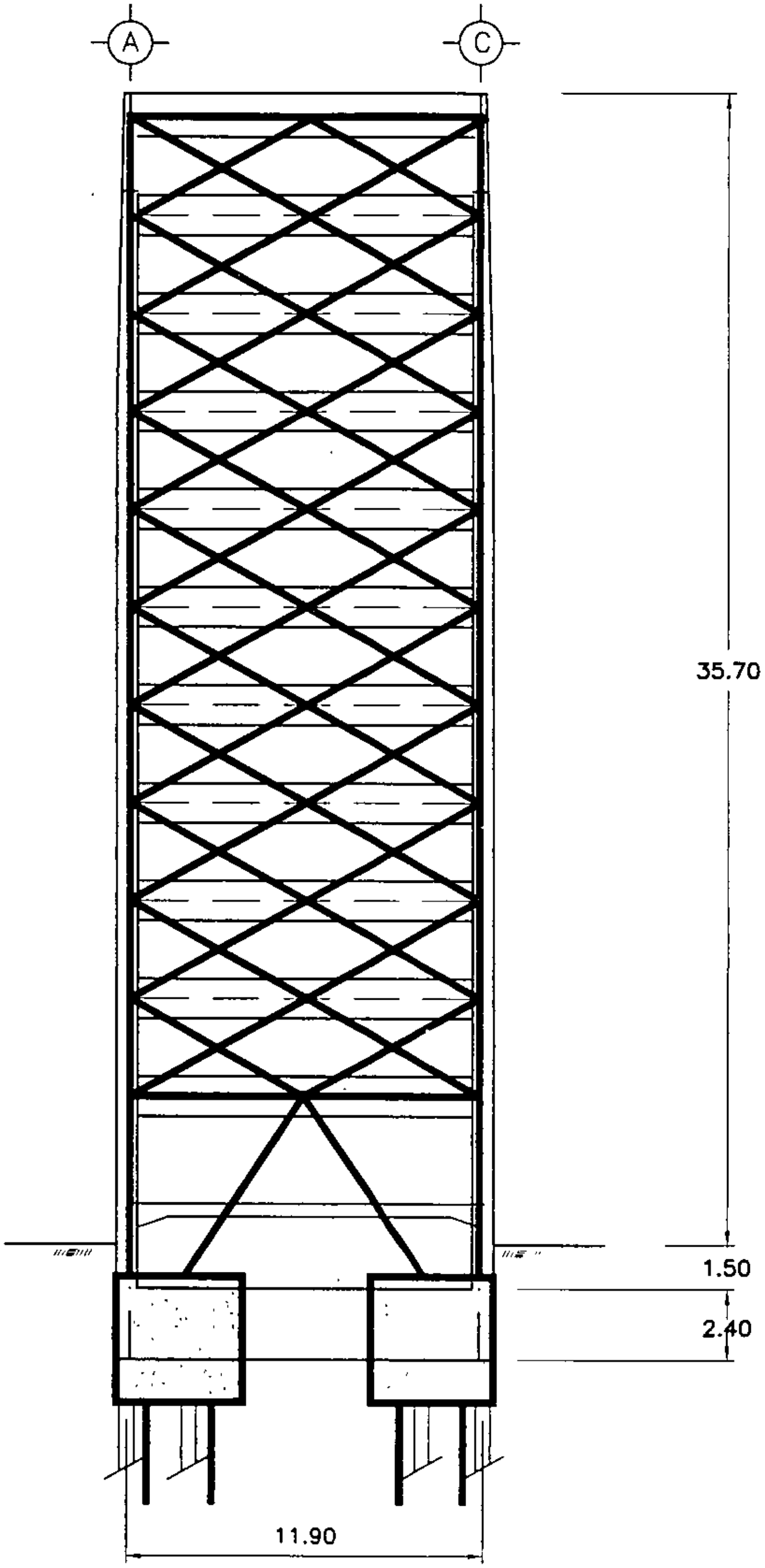
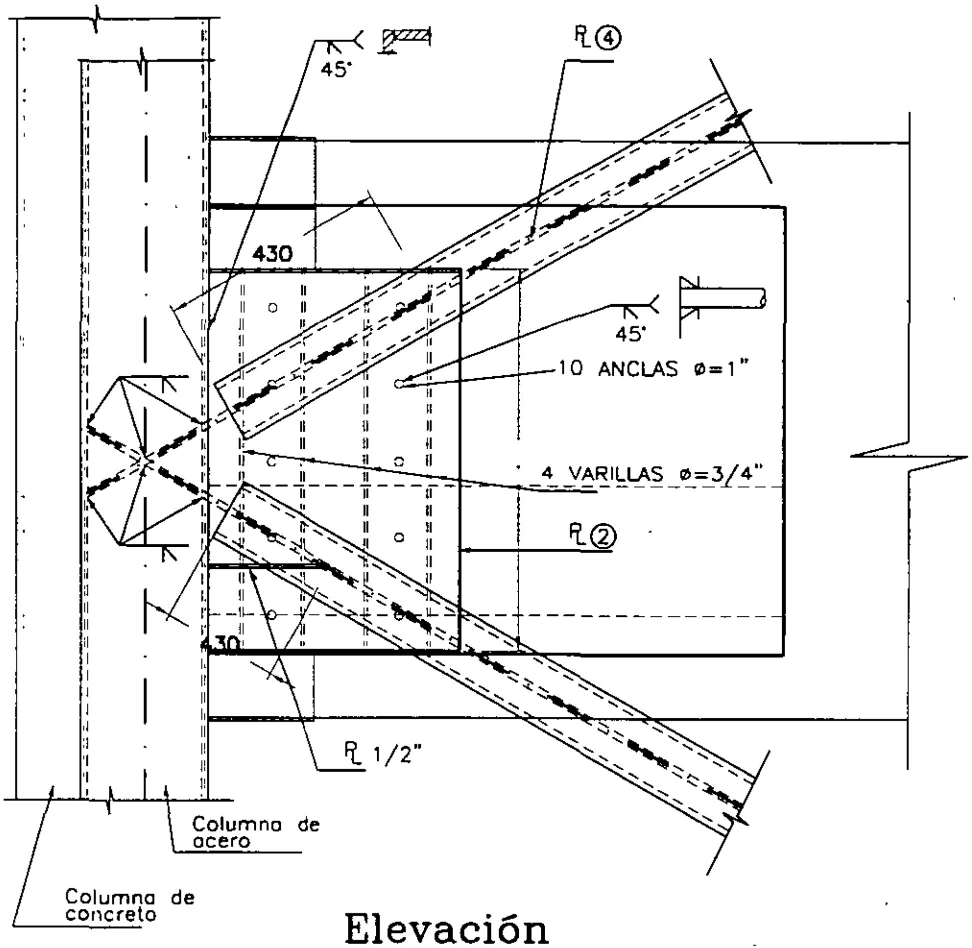
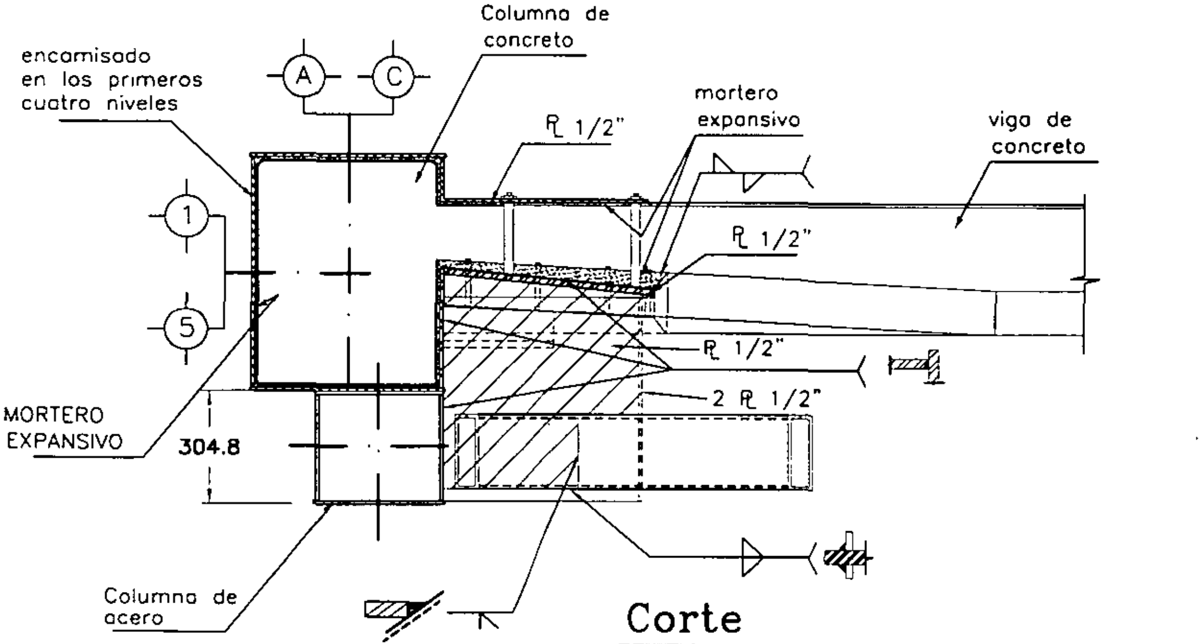


Fig.6 Refuerzos propuestos en marcos 1 y 5.



Elevación



Corte

Fig. 7 Detalles de conexión entre estructura original y los marcos de acero.

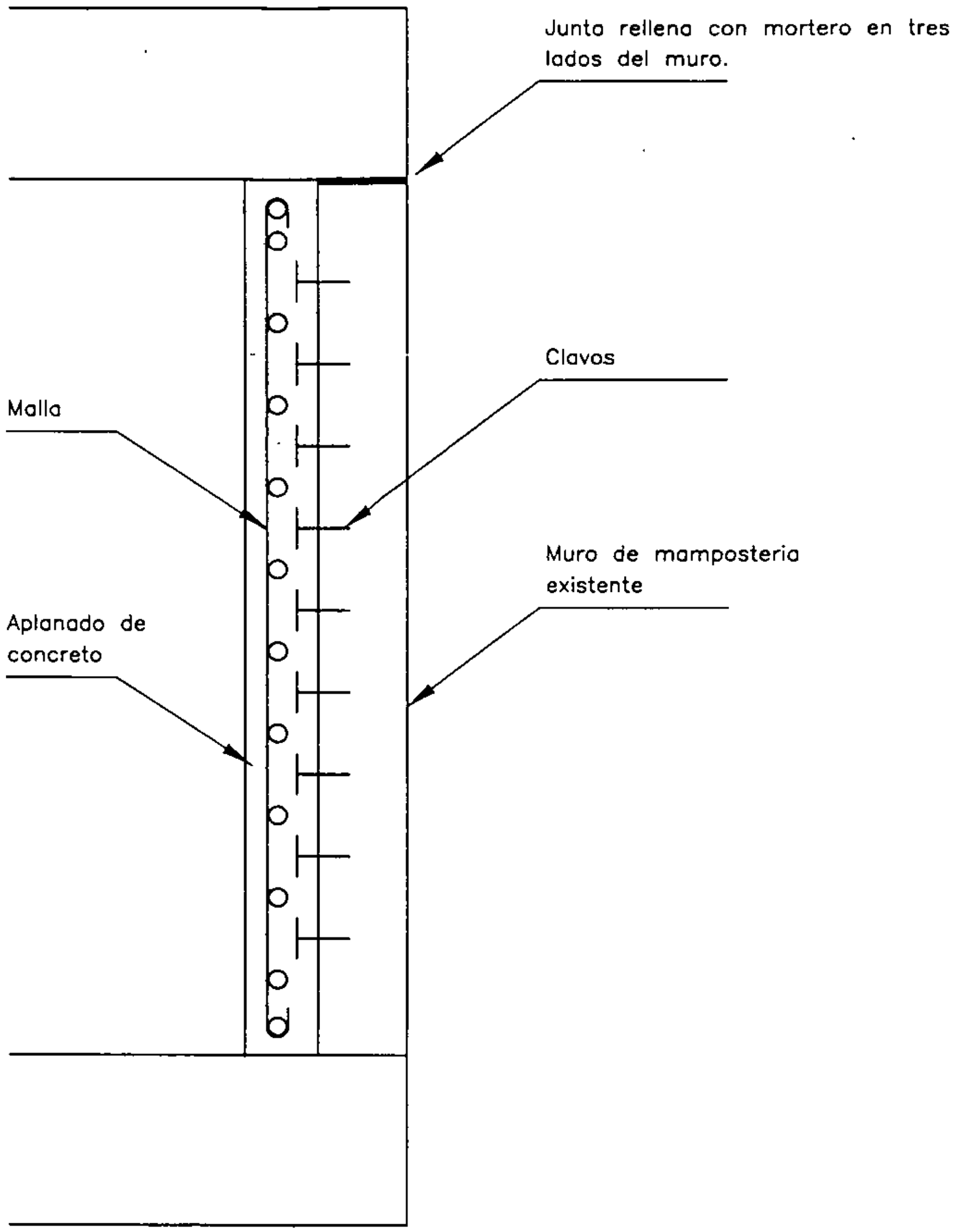


Fig.8 Refuerzo de muros longitudinales de colindancia, marcos A y C.

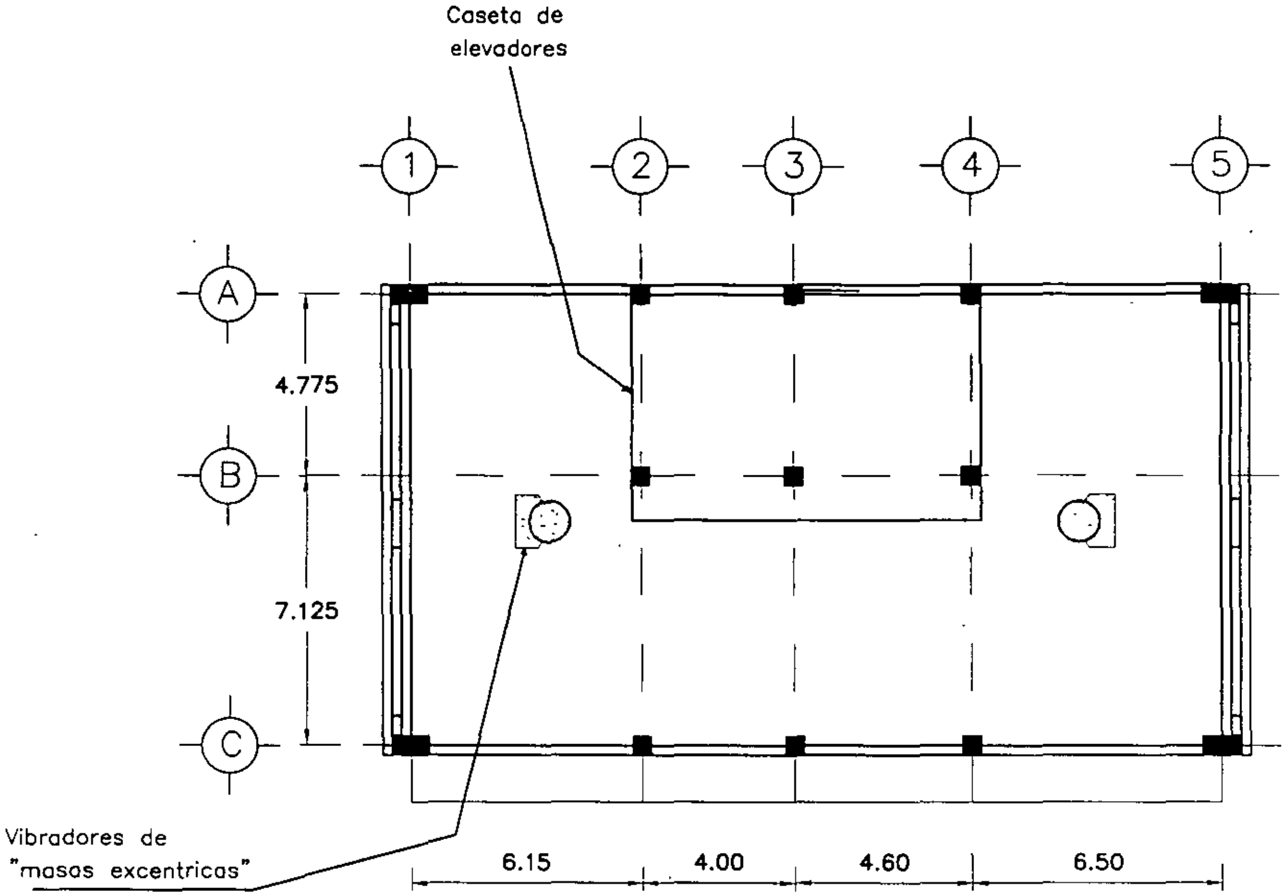


Fig. 10 Planta de azotea mostrando la ubicación de los vibradores.

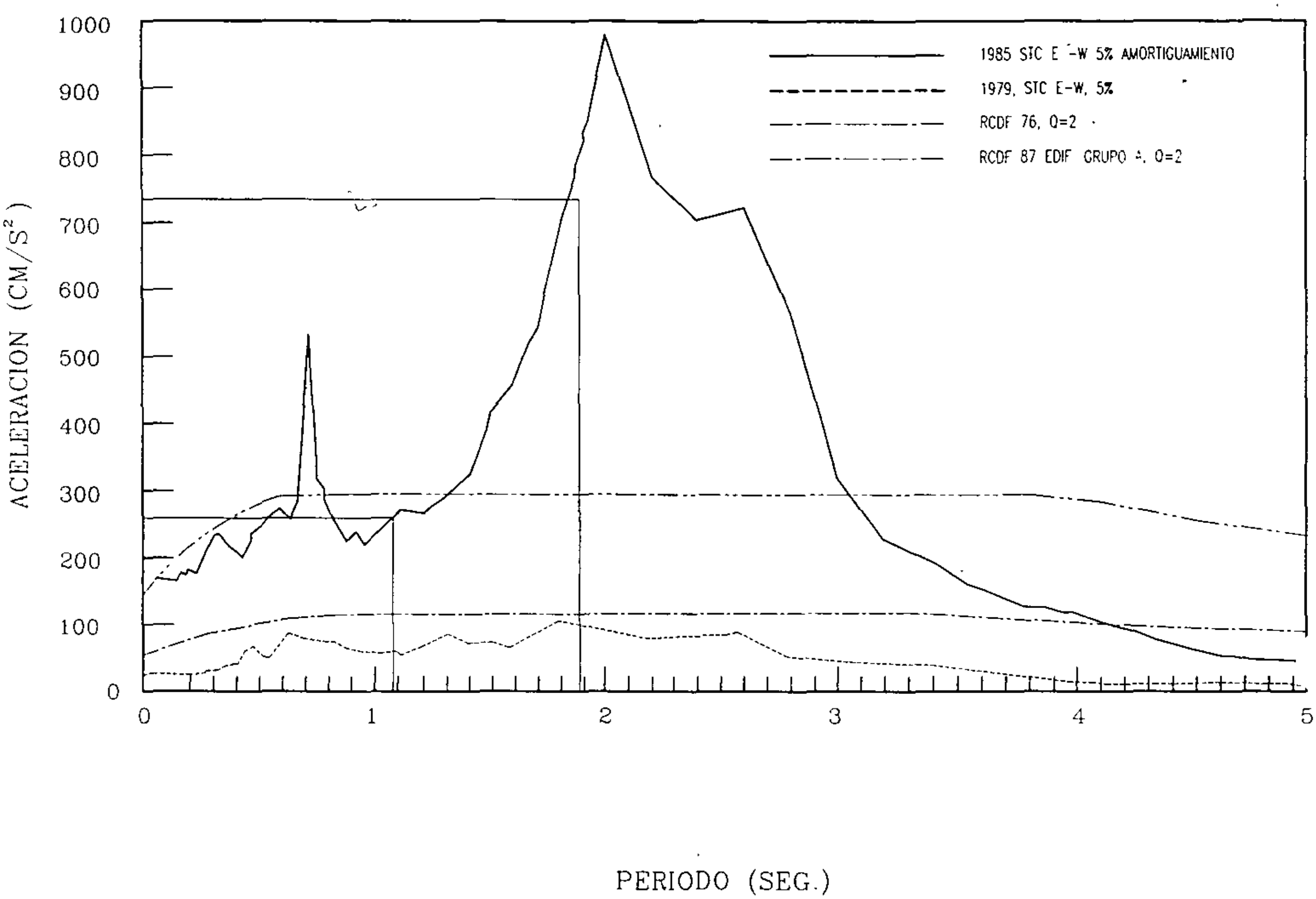
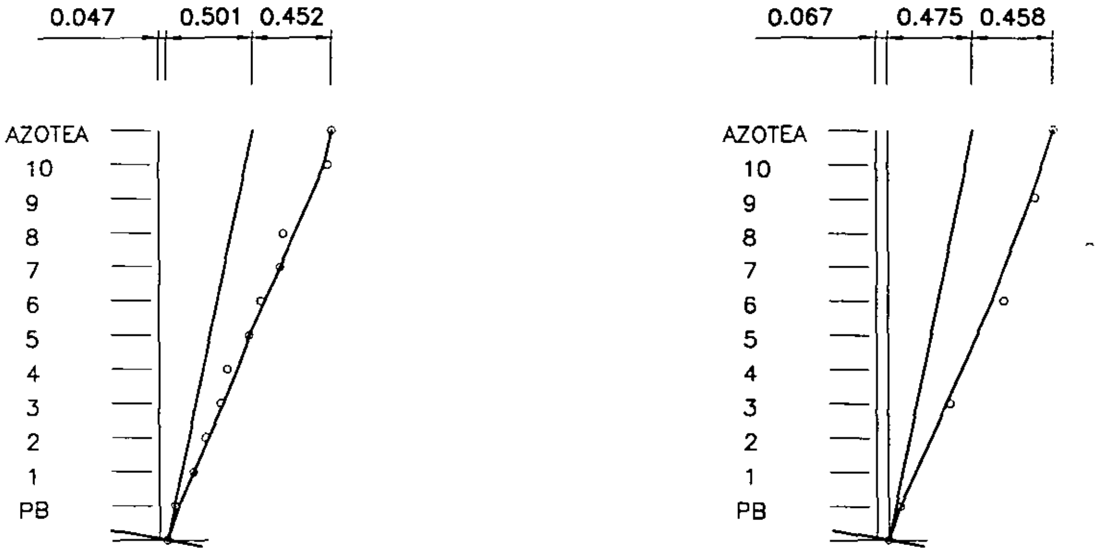


Fig. 9 Espectro de respuesta y de diseño.



PRIMER MODO, E-W		
PERIODO:	1.26	seg.
AMORTIGUAMIENTO:	2.8	%

PRIMER MODO, N-S		
PERIODO:	0.99	seg.
AMORTIGUAMIENTO:	4.8	%

Fig.11 Formas modales obtenidas teórica y experimentalmente.

FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

XXVI CURSO INTERNACIONAL DE INGENIERIA SÍSMICA

MOUDLO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

ALTERNATIVA PARA CONTROLAR LA RESPUESTA SÍSMICA DE LOS EDIFICIOS

EXPOSITOR: DRA. SONIA E. RUIZ GOMEZ
PALACIO DE MINERIA
SEPTIEMBRE DEL 2000

ALTERNATIVAS PARA CONTROLAR LA RESPUESTA SÍSMICA DE LOS EDIFICIOS

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Trabajo de ingreso que presentó la autora el 22 de octubre de 1998 en la Academia Mexicana de Ingeniería como Académica de Número en la Comisión de Especialidad de Ingeniería Civil

INTRODUCCIÓN

En la literatura técnica internacional sobre ingeniería estructural se han propuesto distintas soluciones orientadas a controlar las aceleraciones y los desplazamientos en las construcciones sujetas a la acción de cargas dinámicas producidas por sismo, viento, vibraciones mecánicas, etc. Estas soluciones pueden funcionar en forma pasiva o activa (estas últimas con la ayuda de un sistema de cómputo). Las pasivas se basan principalmente en aumentar el amortiguamiento del conjunto estructural incluyendo la cimentación, en modificar el (los) periodo(s) de vibrar de las estructuras o en una combinación de estas dos acciones. Las activas registran la respuesta del sistema durante un temblor y, con base en esta información, modifican en diversos tiempos los valores instantáneos de las propiedades mecánicas (rigideces, amortiguamientos) de algunos componentes del sistema, o introducen fuerzas externas adicionales, de manera de contrarrestar parcialmente los efectos desfavorables de las perturbaciones naturales que afectan a dicho sistema.

Lo anterior se puede lograr añadiendo a la estructura dispositivos tales como disipadores de energía, amortiguadores de masa resonante, aisladores de base, o combinaciones de éstos.

En la actualidad existe en el mundo un número importante de edificios en los que se han empleado estos dispositivos para controlar la respuesta causada por la acción del viento, por la del sismo, o por otro tipo de solicitaciones. Este trabajo se enfoca a la acción sísmica y a los sistemas de control pasivo. El presente *trabajo* pretende presentar una visión global sobre el tema, con énfasis en los estudios realizados por la autora y su grupo de trabajo.

Muchos profesionales, y sobre todo un gran número de posibles usuarios, no están aún convencidos de que se justifique invertir en estas soluciones innovadoras, sino que prefieren adoptar soluciones tradicionales. Mientras tanto, se continúan realizando estudios en varios países con alto riesgo sísmico (principalmente en Estados Unidos de América, Japón, Italia, Canadá, Nueva Zelanda y China) para tratar de comprender mejor las ventajas y desventajas de estos sistemas.

México ha participado de la inquietud por conocer más sobre este tipo de soluciones, desde hace aproximadamente diez años (Del Valle, 1988). Prueba de ello es que durante el último Congreso Mundial de Ingeniería Sísmica llevado a cabo en 1996 en nuestro país, México ocupó el séptimo lugar en número de artículos presentados sobre el tema (Ruiz, 1996). Por otro lado, también se constató el interés de profesionistas e investigadores mexicanos sobre esta materia durante el Simposio sobre "Disipadores de energía para controlar la respuesta sísmica de edificios" organizado recientemente por la SMIS - Delegación Estado de México.

A continuación se presenta una visión general sobre dos tipos de dispositivos reductores de la respuesta sísmica: disipadores de energía y amortiguadores de masas resonantes.

DISIPADORES DE ENERGÍA SÍSMICA

Tipos de elementos disipadores

Con el fin de controlar la respuesta sísmica se han propuesto diferentes materiales, formas y mecanismos enfocados a aumentar el amortiguamiento de las estructuras. Esto puede lograrse mediante dispositivos que se añaden a la estructura sin incrementar ni su rigidez ni su resistencia total, tales como los disipadores viscosos, o bien mediante dispositivos que, a la vez que aumentan el amortiguamiento, incrementan dichas propiedades mecánicas, como los disipadores histeréticos, los de fricción y los viscoelásticos. Cada uno de estos tipos presenta propiedades específicas que deben considerarse para su diseño.

Los dispositivos de energía de tipo viscoso se caracterizan porque al deformarse desarrollan fuerzas que son proporcionales a su velocidad de deformación. El ejemplo típico es el de un cilindro con un pistón y uno o varios orificios por los que puede escurrir un fluido viscoso. La curva carga- deformación de uno de estos elementos es similar a la de la figura 1a, en donde se observa que la carga adquiere su máximo para el momento en que la deformación es nula, mientras que la primera es nula para cuando la segunda alcanza su máximo. En la figura 1b se muestra la curva carga deformación para un elemento de tipo viscoelástico, caracterizado porque la fuerza que desarrolla consta de dos componentes fuera de fase: una proporcional a la deformación y otra proporcional a la velocidad de deformación; la primera alcanza su máximo cuando la segunda es nula, y viceversa.

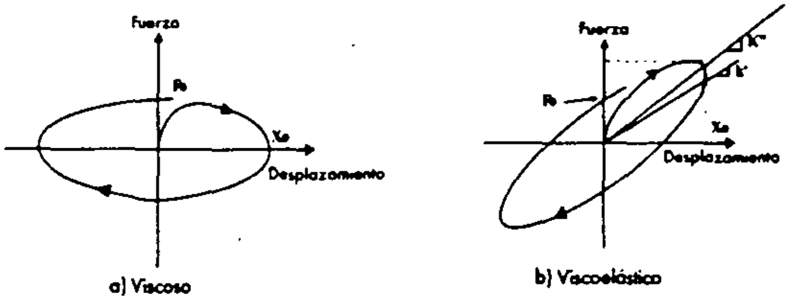
En las figuras 1c y 1d se presentan las curvas carga-deformación de dos elementos disipadores de tipo histerético; la primera de ellas representa el comportamiento de un elemento fabricado con un material capaz de desarrollar comportamiento dúctil. La segunda curva respresenta el comportamiento de una conexión que desarrolla parte de su capacidad mediante fuerzas de fricción: la junta no se deforma mientras la fuerza sobre ella no supere la fuerza de fricción; cuando esto ocurre, la junta se deforma manteniendo constante la carga.

Algunas de las características especiales que se deben considerar para el diseño de los dispositivos antes descritos son las siguientes:

- En relación con los elementos viscosos, debe considerarse que su resistencia se encuentra fuera de fase con su deformación, por lo que los momentos de volteo de la estructura no se incrementan de la misma manera que ocurriría si se tratase, por ejemplo, de disipadores histeréticos.

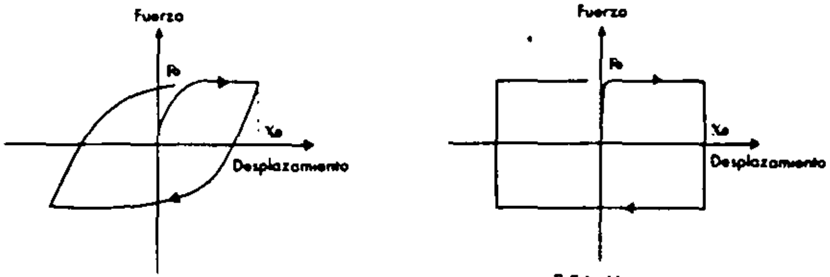
- Para los disipadores viscoelásticos debe considerarse en forma especial que su comportamiento depende de la frecuencia de la excitación, de la temperatura y del nivel de su deformación por cortante. Estos disipadores absorben energía aún cuando sus deformaciones relativas sean muy pequeñas.

- Los disipadores basados en la fluencia de metales (como el acero y el plomo) sólo disipan energía después de que se alcanza su límite de fluencia. Para que estos disipadores trabajen en forma eficiente es necesario que sus deformaciones sean entre moderadas y altas (por ejemplo,



a) Viscoso

b) Viscoelástico



c) Deformación plástica de metales

d) Fricción

Figura 1.- Comportamiento carga -deformación idealizado de diferentes tipos de disipadores de energía.

el edificio en donde se coloquen, presente deformaciones grandes de entrepiso). La energía que disipan estos elementos se transforma en calor por el trabajo mecánico de placas o elementos sujetos a flexión, torsión, rolado por flexión, etc. Generalmente este tipo de disipadores puede desarrollar un gran número de ciclos ante cargas alternadas sin que se degrade ni su resistencia ni su rigidez.

- Para los disipadores de fricción se debe tomar en cuenta que debido a los cambios bruscos de su rigidez se pueden llegar a excitar modos superiores de vibración. Los disipadores de este tipo generalmente no actúan ante temblores de intensidad pequeña y moderada.

Experiencia en México

En nuestro país se han reforzado varios edificios con disipadores histeréticos de energía tipo ADAS (Martínez Romero, 1993), y uno con placas de acero sometidas a fricción (Sánchez Martínez, 1993). Recientemente se construyó un edificio de acero con disipadores viscoelásticos (Miranda, 1998) en Santa Fe, al oeste de la ciudad de México, y además, dos torres altas con disipadores histeréticos (ADAS) en Acapulco, Gro.

En una revisión bibliográfica realizada recientemente por la autora (Ruiz, 1998) se menciona que la mayor parte de los estudios realizados en nuestro país sobre disipación de energía sísmica son de tipo analítico y que se han dedicado muy pocos esfuerzos a la parte experimental. Los estudios de tipo analítico tratan principalmente sobre disipadores pasivos de energía de tipo histerético (ADAS, TADAS y soleras en forma de U).

En México se ha analizado la respuesta estructural de varias decenas de edificios con disipadores histeréticos y sin éstos, con periodos comprendidos entre 0.75 y 2.5s (Tena Colunga, 1998, Ruiz, 1998, etc). La mayor parte de los análisis se refieren a edificios ubicados en el terreno blando de la ciudad de México, y sólo algunos de ellos, a edificios en terreno de transición. Frecuentemente se emplea como excitación de diseño la componente Este-Oeste del registro obtenido en la Secretaría de Comunicaciones y Transportes el 19 de septiembre de 1985 (SCT-1985). Es deseable ampliar estos estudios para otras condiciones de terreno local.

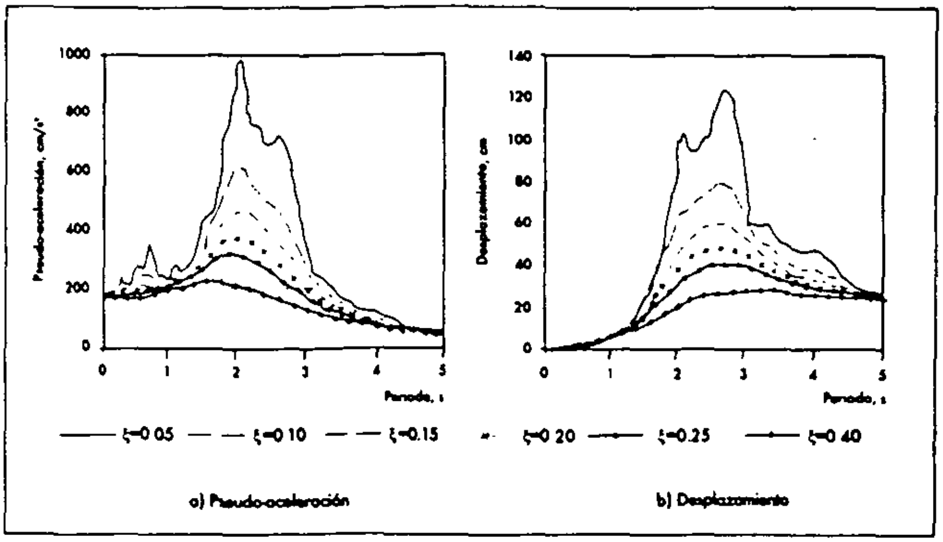
Variables que afectan la eficiencia de los elementos disipadores de energía

La reducción en la respuesta máxima que proporciona el incremento de amortiguamiento en las estructuras se puede vizualizar de manera simplificada a través de los espectros de aceleración y desplazamiento. Supónganse, por ejemplo, los espectros de pseudo-aceleraciones y de desplazamientos del registro SCT-1985 lineal ($\mu = 1$) y no lineal ($\mu = 2$, con una rigidez de postfluencia igual a 3% de la rigidez inicial) para diferentes amortiguamientos críticos, 5, 10, 15, 20, 25 y 40%, como los que muestran las figuras 2 y 3. En la primera de éstas se puede observar que las aceleraciones lineales ($\mu = 1$) se reducen considerablemente al incrementar el amortiguamiento de 5 a 25% dentro del intervalo de periodos comprendido entre 1.5 y 3s, aproximadamente. Algo similar sucede cerca del periodo 0.7s. Sin embargo, se puede apreciar que fuera de estos intervalos la reducción de la aceleración, y por lo tanto, de la fuerza cortante basal, es poco significativa. Asimismo, se puede observar de la figura 2b que a mayor amortiguamiento los desplazamientos máximos son menores, especialmente dentro del intervalo entre 1.8 y 4.0s. El mismo ejercicio puede hacerse para los sistemas que presentan un comportamiento no lineal moderado ($\mu = 2$). También en este caso (figuras 3a y b) el efecto en la reducción de la respuesta es mucho menor cuando el amortiguamiento crítico aumenta de 25 a 40%, que cuando pasa de 5 a 25%.

Lo anterior constata el hecho conocido de que introducir amortiguamiento viscoso en las estructuras da lugar a una respuesta estructural menor. La decisión de introducirlo en una construcción dependerá principalmente de un análisis costo-beneficio.

La interpretación sobre la adición de amortiguamiento no viscoso en las estructuras se puede hacer de manera similar; sin embargo, en este caso se debe tomar en cuenta que algunos tipos de disipadores introducen, además de amortiguamiento,

Figura 2.- Espectros lineales del registro SCT-85 para diferentes amortiguamientos.



a) Pseudo-aceleración

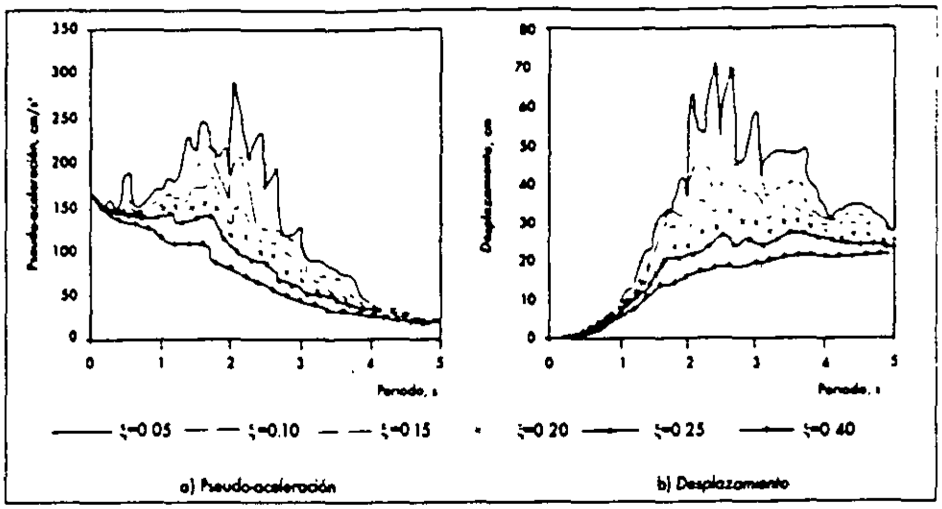
b) Desplazamiento

una rigidez adicional a la estructura, y por lo tanto hacen que su período de vibración disminuya.

Para explorar la conveniencia de reforzar un edificio con contravientos o con disipadores de energía histeréticos se puede seguir un razonamiento similar al de los párrafos anteriores. Para ilustrar esto en seguida se mencionan los resultados de un estudio realizado recientemente sobre un edificio en el que se deseaba saber si era más conveniente reforzarlo con contravientos o con disipadores de energía (Limón y Ruiz, 1997). Su período inicial es de 2s. El sismo de diseño se consideró igual al del registro SCT-1985. En este caso se demostró que el edificio con contravientos presentaba un comportamiento casi lineal y que su período fundamental de vibración era de 1.3s También se calculó que el edificio desarrollaba un cortante sísmico basal igual a 0.24 ante la acción del registro mencionado. Estas cifras pueden verse de manera aproximada en la figura 2a, correspondiente al espectro lineal de aceleraciones SCT-1985.

Como solución alternativa, se consideró el edificio reforzado con disipadores de energía. Se ensayaron tres diferentes distribuciones espaciales de los disipadores, considerando diferentes tipos de ellos (TADAS y soleras en U). Los promedios de las demandas de ductilidad globales de los tres edificios reforzados con disipadores resultaron iguales a 1.12, 2.57 y 2.62, y los coeficientes sísmicos basales iguales a 0.23, 0.22 y 0.23. Los periodos de vibración efectivos (tomando en cuenta las reducciones de rigidez asociadas con el comportamiento no lineal de los sistemas) en los edificios con disipadores se encuentran entre 1.33 y 2s. En la figura 3a (μ= 2) se puede comprobar que para un periodo de 1.5s la ordenada espectral correspondiente, para un amortiguamiento crítico de 5%, es de 0.22 aproximadamente.

Los resultados del análisis hacen ver que en estos casos los cortantes basales del edificio tienen valores similares cuando se utilizan contravientos y cuando se usan disipadores. Sin embargo, es obvio que el refuerzo con contravientos tiene un costo de construcción menor que la solución con disipadores.



a) Pseudo-aceleración

b) Desplazamiento

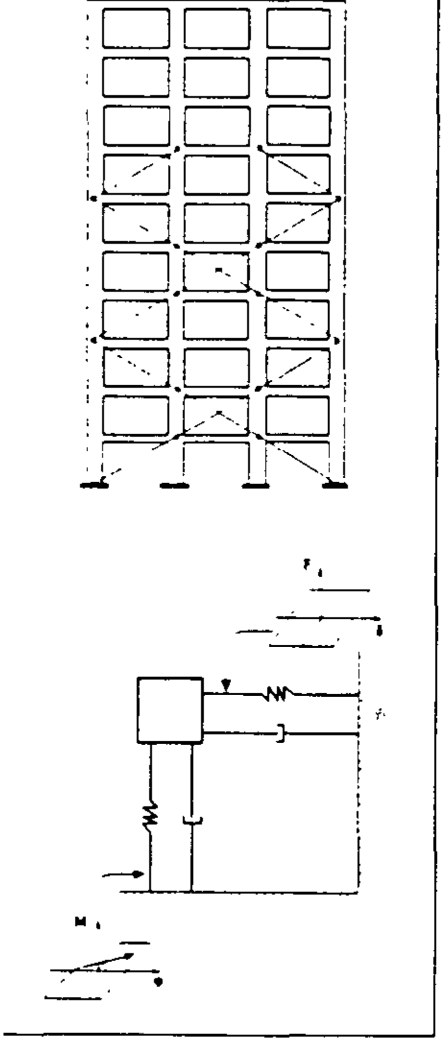
Figura 3.- Espectros no lineales del registro SCT-85 para diferentes amortiguamientos demanda de ductilidad μ=2

Por otro lado, en la figura 2b puede verse que los desplazamientos del sistema para un periodo de 1.3s y μ = 1 son menores que los de la estructura con un periodo de 1.5s y μ=2 (figura 3b), ambas con amortiguamiento crítico de 5%. Esto concuerda con los desplazamientos obtenidos para el edificio de múltiples niveles y crujías reforzado con contravientos (figura2b) o con disipadores (figura 3b).

Con un análisis similar al anterior se puede hacer ver que, en el caso de que la estructura por reforzar tuviera un periodo muy largo, tal que se encontrara muy a la derecha de la zona descendente del espectro, los cortantes basales resultarían mayores para la estructura reforzada con contravientos que para la reforzada con disipadores En este caso los momentos de volteo y las fuerzas axiales sobre la cimentación serían más grandes para el caso con contravientos, por lo que la mejor solución sería utilizar disipadores (Martínez Romero, 1993; Gómez et al, 1993).

Estudios sobre modelos simplificados

Los razonamientos de los párrafos precedentes se basan en sistemas muy sim cados de un grado de libertad, ei que no se toman en cuenta variables tales como sobre-resistencia, degradación de las propiedades mecánicas de la estructura que contiene a los disipadores, etc. En la literatura se han explorado varias formas de representar el comportamiento no lineal de edificios mediante sistemas simplificados de un grado de libertad (1gdl). El grupo de trabajo supervisado por la autora actualmente calibra un método basado en un análisis estático no lineal ("push over") para relacionar la respuesta no lineal de edificios con disipadores con la de sistemas equivalentes de 1gdl constituidos por una masa y dos elementos estructurales El que representa al edificio se degrada ante cargas cíclicas, mientras que el que representa al sistema disipador no se degrada (figura 4). Los resultados obtenidos (Badillo et al, 1998) hacen ver que es posible representar de forma aproximada el comportamiento no lineal de edificios con disipadores a través de sistemas sirr cados de 1gdl. Algunos resultados tos estudios se presentan en las fi



jura 4 · Marco con disipadores y su correspon-
nte sistema equivalente de lgdl

5a y b. En éstos se muestran el desplazamiento máximo obtenido en la azotea de un edificio de diez niveles con disipadores (SVGDL) y el del sistema de 1gdl equivalente (figura 5a), así como la distorsión máxima de entrepiso (figura 5b). Las estructuras se someten a la acción de 29 sismos simulados a partir del registro SCT-1985. Se observa que los resultados del sistema simplificado son aceptables.

Obviamente, los resultados de un sistema de 1gdl no proporcionan información sobre otros aspectos importantes de diseño, como por ejemplo la distribución espacial óptima de los disipadores en un edificio. Para ello es necesario llevar a cabo estudios de estructuras de múltiples grados de libertad en dos o en tres dimensiones. Para estas últimas es importante analizar el efecto de torsión causado por excentricidades provocadas por los disipadores, por ejemplo, por una instalación defectuosa, por defectos de construcción de los mismos o por falta de mantenimiento. Este tema amerita mayor estudio, ya que ha sido muy poco explorado, tanto en el medio internacional como en el nuestro.

Con respecto a la distribución de disipadores se analizó la respuesta de un marco estructural plano de diez niveles con disipadores colocados según ocho arreglos espaciales distintos (ver figura 6). Los autores (Urrego, Ruiz y Silva, 1993) hacen ver que los arreglos más convenientes son aquéllos en que las diagonales están dispuestas de manera de evitar la ocurrencia de grandes fuerzas axiales sobre las columnas que transmiten los momentos de volteo a la cimentación. En la figura 6 se muestran los ocho arreglos. De éstos, los más convenientes son los indicados como "Marco 0" y "Marco 1".

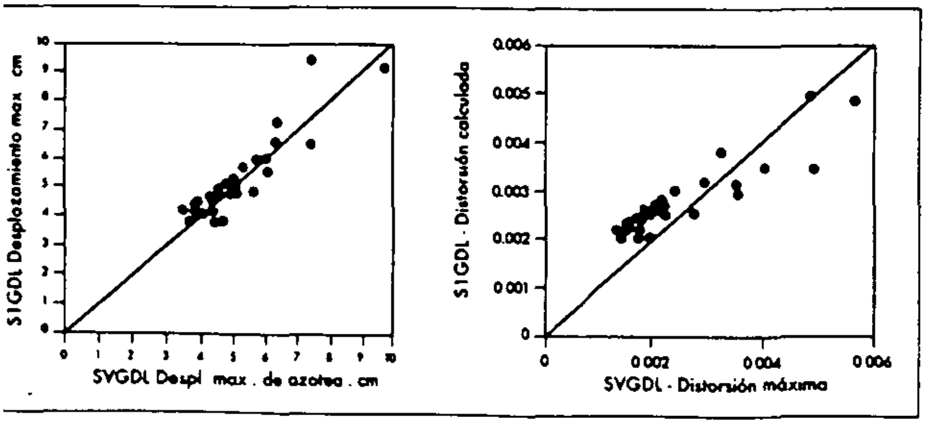
Criterios de diseño sísmico

Los criterios y métodos de análisis y diseño de edificios con disipadores en general son más complicados que los de los edificios convencionales. Para el análisis de los primeros generalmente se parte de un prediseño, el cual se afina mediante iteraciones realizadas con programas de análisis no lineal en el tiempo (Silva y Ruiz, 1993). Para el diseño final se toman en cuenta básicamente las siguientes condiciones:

- Que los disipadores absorban eficientemente la energía y cumplan la función para la que fueron diseñados. En el caso de disipadores histeréticos se debe cuidar que las demandas de ductilidad de estos estén dentro de los límites de aceptabilidad establecidos a partir de resultados de pruebas de laboratorio.

- Que las demandas de ductilidad globales y locales no excedan las capacidades de la estructura. La demanda de ductilidad global en general se adopta del orden de 2. Un criterio alternativo es que las trabes sean capaces de desarrollar las rotaciones plásticas demandadas. Alternativamente, se puede diseñar la estructura de modo que desarrolle un comportamiento lineal y los disipadores presenten uno no lineal.

- Que los desplazamientos relativos de entrepiso del edificio con disipadores no sean mayores que los valores asociados a estados límite de servicio o de capacidad última. Estos pueden ser recomendados



jura 5.- Reespuestas de sistema de un edificio de diez niveles con disipadores y su sistema
uivalente de Lgid con disipadores

por algún reglamento de construcciones o de rehabilitación de edificios. En el caso del Reglamento de Construcciones del Distrito Federal (RCDF-1993) el límite de desplazamiento (asociado a la capacidad última) para estructuras esqueletales con muros desligados, es de 0.012 de la altura del entrepiso.

- Que se cumpla con la normatividad sísmica del lugar donde se ubique la construcción. La mayoría de los reglamentos de diseño en el mundo permiten utilizar dispositivos reductores de la respuesta sísmica con la condición de que los diseños sean aprobados por las autoridades competentes (Ruiz y Alvarez, 1995). En el RCDF-1993 se indica que se pueden emplear criterios de diseño diferentes a los especificados en su capítulo III y en las Normas Técnicas Complementarias si se justifica, a satisfacción del Departamento del Distrito Federal, que los procedimientos de diseño empleados dan lugar a niveles de seguridad no menores que los que se obtengan empleando los previstos en ese Ordenamiento. Tal justificación debe realizarse previamente a la solicitud de la licencia de construcción.

Estudios costo-beneficio

La decisión de reforzar con disipadores o con contravientos, o de diseñar un edificio nuevo en forma convencional o con disipadores, no sólo es un problema de diseño estructural, depende en gran medida de analizar si se justifica cierta inversión extra a cambio de mayor seguridad y/o comodidad para los ocupantes del inmueble durante su vida útil.

En México se han formulado criterios para diseño, reparación y mantenimiento de sistemas con disipadores basados en la minimización de la suma de los costos iniciales, de daño y de mantenimiento (Esteva, Díaz y García, 1998). Por otro lado, se han realizado algunos análisis de costos de construcción de edificios con disipadores. Por ejemplo, Ruiz et al (1995, 1996) informan que el costo inicial de un edificio nuevo de diez niveles con disipadores, diseñado para un sismo de diseño como el SCT-85, es 3.5% mayor que el de un edificio similar (con el mismo período de vibración) diseñado en forma convencional, de manera que presente el mismo nivel de daño. Cuando se trata de edificios de 20 niveles, dicha diferencia es aproximadamente de 9.5%. Sin embargo, es muy probable que si se tomaran en cuenta, no solamente los costos de construcción, sino los costos totales durante la vida útil de la construcción, resultaría más conveniente la solución con disipadores de energía.

Comentarios finales sobre sistemas con disipadores de energía:

En general, se puede decir que los disipadores de energía son una solución factible de aplicar tanto en edificios nuevos como para el refuerzo de edificios existentes. Como se mencionó antes, la eficiencia de estos dispositivos depende, entre otras variables, de la relación entre los períodos naturales de la estructura y los dominantes de los movimientos que se esperan en el sitio de interés. La decisión de utilizar uno u otro dispositivo, o bien de elegir una solución convencional, se debe basar tanto en un análisis de costos totales (que incluya costos de mantenimiento, reposición, daños estructurales y no estructurales, pérdida de funcionalidad, etc.) como en un análisis de la respuesta de la estructura ante las excitaciones esperadas durante su vida útil, que pueden asociarse a diferentes estados límite (de servicio o de capacidad última).

AMORTIGUADORES DE MASAS RESONANTES

Otra manera de controlar la respuesta dinámica de las estructuras consiste en añadirles uno o varios sistemas de amortiguadores de masas resonantes (AMR). Cada uno de éstos está constituido básicamente por una masa, un resorte y un amortiguador. Dicha masa puede ser sólida o líquida (en ese caso se le llama amortiguador de líquido resonante). El principio en el que se basan estos sistemas es que los periodos de vibrar de la estructura principal se

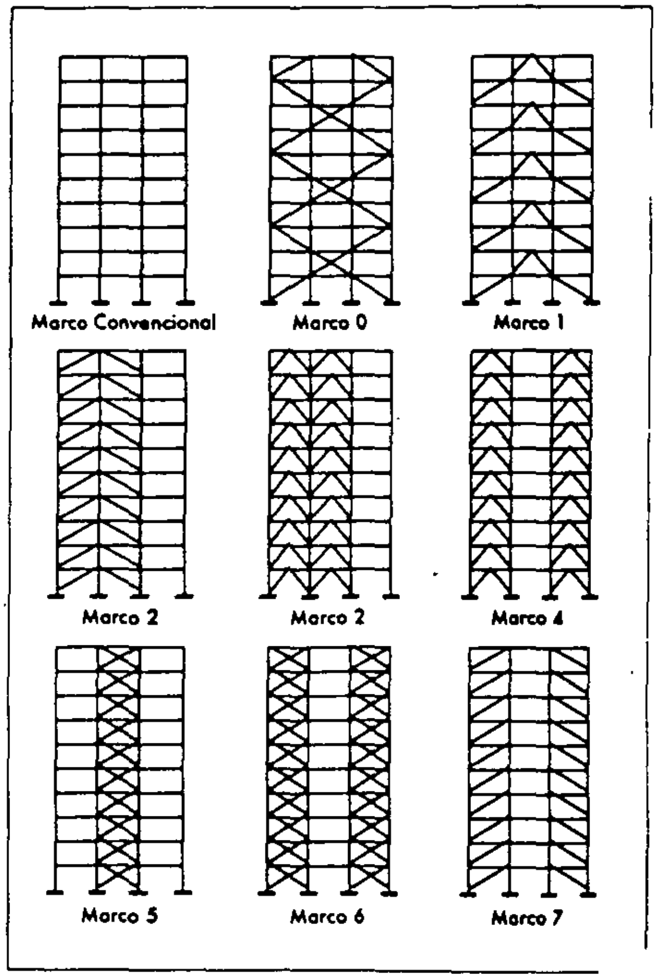


Figura 6.- Marcos con diferentes arreglos de disipadores

sintonicen con el de las masas resonantes, de manera que cuando cierto período se excite, el dispositivo entre en "resonancia" fuera de fase con el movimiento de la estructura, lo que hace que se reduzca la respuesta de ésta. Para controlar la respuesta asociada al modo fundamental de vibración, es común colocar un AMR (con masa líquida o sólida) en la azotea del edificio en cuestión

Algunas aplicaciones de los AMR

Los AMR se han utilizado en forma eficiente para controlar la vibración de sistemas mecánicos, puentes, chimeneas, torres y antenas. También se han aplicado con éxito para controlar las vibraciones causadas por viento sobre edificios; sin embargo, no existe consenso entre investigadores e ingenieros estudiosos de este tema en cuanto a su efectividad para reducir la respuesta de edificios ante acciones sísmicas.

Algunos ejemplos de aplicación para controlar la respuesta ante la acción del viento son los siguientes: a) la Torre John Hancock de Boston, que cuenta con 60 pisos. A ésta se le añadieron dos AMR que pesan en total 600 ton, b) el Centro Citicorp en Manhattan al cual se le añadió una masa de 400 tons (cerca del 2% de su peso total) en su piso número 63. El desplazamiento máximo relativo que se espera en dicho amortiguador con respecto a su punto de sujeción a la construcción es de 1.4m; c) la antena de la Torre Nacional Canadiense, que tiene 102 m de altura. Los amortiguadores de ésta se sintonizaron de manera de reducir su segundo y su cuarto modos de vibración; d) la Torre del Puerto de Chiba en Japón, con 125 m de altura; y e) la Torre de Cristal localizada en Osaka, Japón, a la cual se añadieron varios AMR en forma de péndulo. Esto se logró colgando los equipos de enfriamiento con cables de 4m de largo; además, se colocaron amortiguadores de aceite para introducir amortiguamiento viscoso al sistema. En México no se han empleado dispositivos de este tipo en edificios.

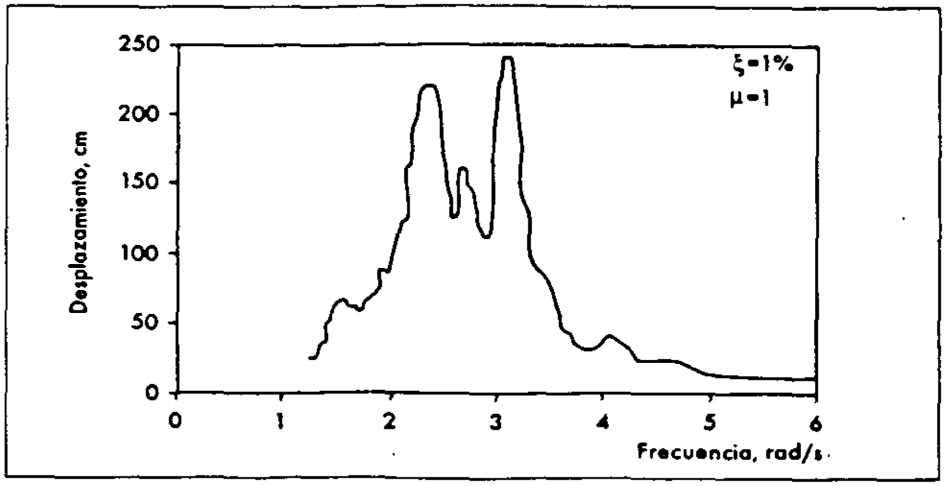
Variables que afectan la eficiencia de los amortiguadores de masa resonante

Cuando a un sistema lineal se añade un AMR, diseñado con el objetivo de absorber vibraciones, en general el conjunto resulta con un período fundamental de vibración mayor que el del sistema sin apéndice. Por otro lado, aparecen en el conjunto modos de vibrar nuevos debido a la presencia del AMR.

Los desplazamientos máximos finales del sistema lineal dependen en gran medida de las ordenadas espectrales de desplazamientos del sismo de diseño correspondientes a las frecuencias de vibrar del conjunto, así como de sus respectivos factores de participación y de la duración de la excitación (Esteva y Ruiz, 1997).

Para ilustrar lo anterior supóngase un sistema de un marco de un nivel y una crujía al que se le añade un AMR en su parte superior, con lo que se obtiene un sistema de dos grados de libertad. La excitación de diseño es el registro SCT-1985, cuyo espectro de desplazamientos para 1% de amortiguamiento se muestra en la figura 7. El marco original tiene una frecuencia fundamental de 2.20 rad/s, que coincide aproximadamente con un pico del espectro. La masa del apéndice es 0.04 veces la del marco principal, y los períodos de vibración de ambos sistemas son iguales. Las primeras dos frecuencias naturales de vibrar del sistema con AMR son iguales a 1.99 y 2.43 rad/s, y las correspondientes formas modales son (1, 5.52) y (1, -4.53). De la figura 7 se deduce que las ordenadas del espectro de desplazamiento son 220cm para el sistema principal, 85 y 175cm para la primera y segunda frecuencias. Los correspondientes factores de participación modal son 0.55 y 0.45. Aplicando la regla de la doble suma cuadrática de Newmark y Rosenblueth (1971) se obtiene que el desplazamiento relativo es 104.1cm, que es aproximadamente 47% de 220cm. Con este ejemplo se muestra que tanto los factores de participación modal como los valores de las ordenadas espectrales para los periodos de vibración de los sistemas original y modificado, juegan un papel muy importante en los resultados finales. También este ejemplo pretende ilustrar que el AMR es eficiente en casos en donde el período de vibrar del sistema principal coincide con el período dominante de la excitación. La eficiencia es mayor cuando ésta es de banda estrecha.

Figura 7.- Espectro lineal de desplazamiento del registro SCT-1985



También la duración de la excitación tiene un papel importante en la reducción de la respuesta, ya que mientras más larga sea dicha duración, las respuestas asociadas con modos de vibración con periodos cercanos, más tienden a desacoplarse. Esto se puede deducir a partir de analizar la regla de Newmark y Rosenblueth (1971) antes mencionada.

Por otro lado, para el diseño de estos dispositivos se debe considerar que su eficiencia es muy sensible a la relación entre su periodo y el de la estructura principal. La reducción de la respuesta puede presentar grandes desviaciones debido a pequeñas variaciones de las propiedades mecánicas de las partes que componen el conjunto, o debido a las incertidumbres inherentes del movimiento sísmico que se espera en el sitio en donde se desplanta la estructura.

La influencia de esta última variable fue estudiada por Suárez, Ruiz y Esteva (1993). Los autores analizan la respuesta probabilista de sistemas estructurales lineales provistos de AMR's cuando se consideran diferentes valores de las relaciones de periodos y masas. En el estudio no se consideran las incertidumbres en las propiedades de los sistemas principales. Para el análisis se considera un conjunto de sismos simulados a partir del registro SCT-1985 (Grigoriu, Ruiz y Rosenblueth, 1988). Los autores concluyen que los coeficientes de variación que se esperan en las relaciones de desplazamientos máximos de sistemas con AMR entre los correspondientes a sistemas sin AMR (D_{CA}/D_{SA}) son del orden de 20 a 30%. En las Figuras 8a y b se ilustran algunos resultados cuando los periodos de los sistemas son iguales a 2 y a 2.5s, y las relaciones entre la masa del AMR y la del sistema principal son de 0.01, 0.03 y 0.05. Se puede fácilmente observar que la eficiencia del sistema disipador es mucho mayor para el sistema con periodo de 2s, que coincide con el periodo dominante de la excitación, que cuando el periodo es de 2.5s.

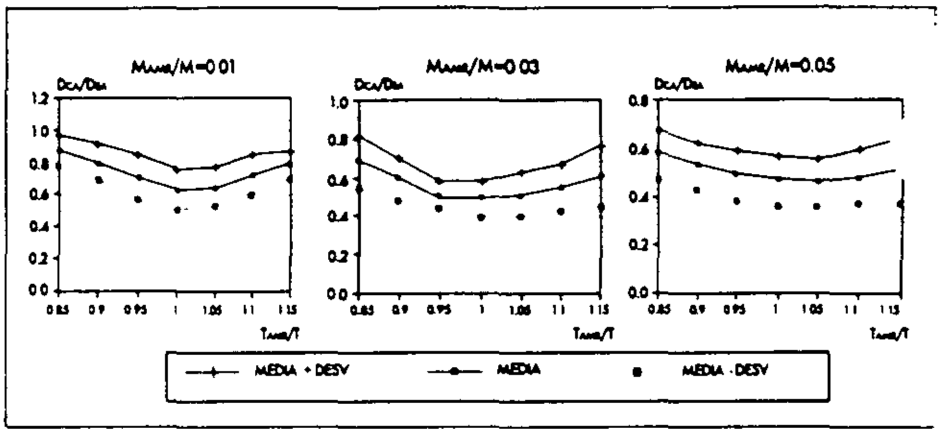


Figura 8a.- Desplazamiento normalizado medio, m ás y menos una desviación estándar T=2.0s

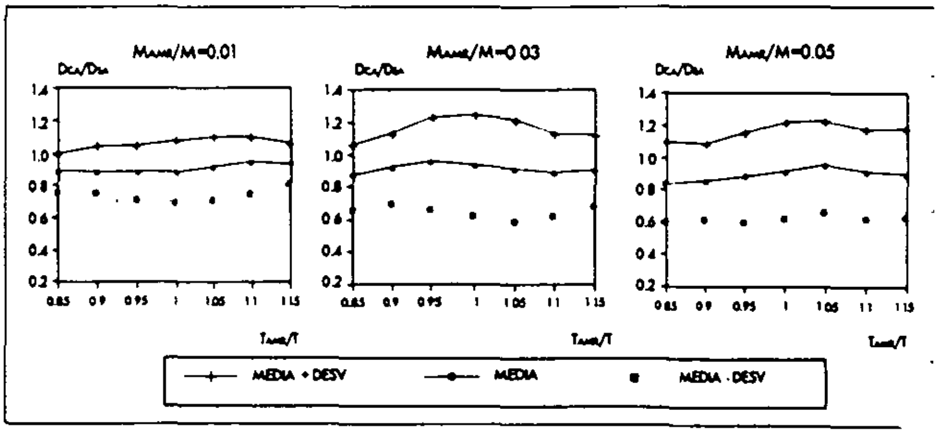


Figura 8b.- Desplazamiento normalizado medio, más y menos una desviación estándar. T=2.5s

Una variable más que influye en la respuesta de un sistema es el amortiguamiento del AMR. Algunos autores (Villaverde y Koyoama, 1993) opinan que la adición de un AMR con alto amortiguamiento (y alta relación de masas) aumenta el amortiguamiento del primer modo de vibrar de la estructura; como consecuencia, se pueden reducir los desplazamientos de entrepiso.

Sin embargo, en un trabajo publicado recientemente (Soto Brito y Ruiz, 1998) se encontró que esto no necesariamente es cierto. En dicho estudio se analizó la respuesta de un edificio de 22 niveles y cuatro crujías, sujeto a la acción de los sismos SCT-1985 y SCT-1989. El periodo del AMR se hizo coincidir con el del edificio. La relación de masas se adoptó igual a 0 03 Se varió el amortiguamiento del AMR entre 5 y 30% del crítico. Los resultados de los desplazamientos de entrepiso (ver figura 9) indican que dicha respuesta no siempre es menor cuando el amortiguamiento del AMR aumenta. Sin embargo, el desplazamiento del AMR con respecto a su base (azotea del edificio) sí se reduce cuando se incrementa su amortiguamiento.

Esto representa una gran ventaja, ya que uno de los principales problemas de los

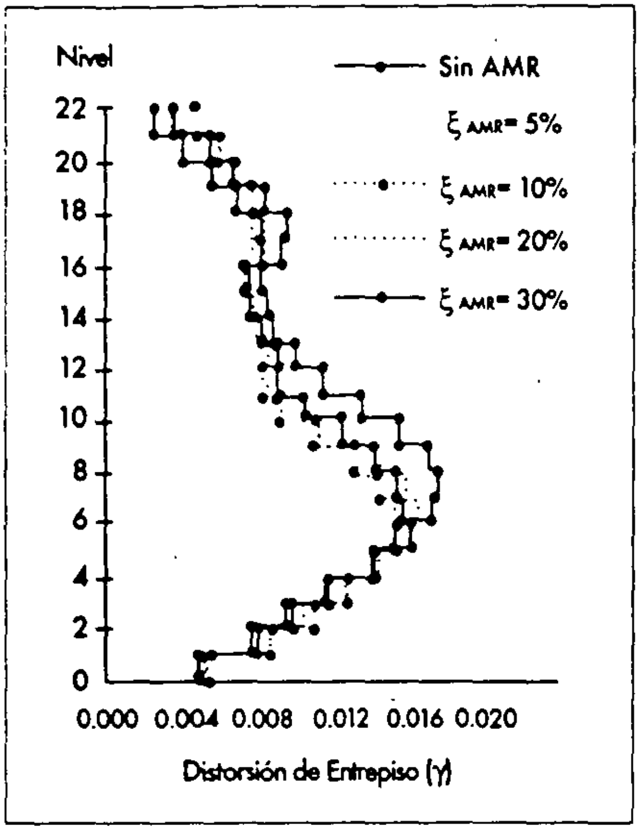


Figura 9 · Envolventes de distorsión de entrepiso $M_{AMR}/M=0.03$, $T_{AMR}/T=1.0$

AMR es que desarrollan desplazamientos relativos muy elevados con respecto a su apoyo. Por ejemplo, para el sismo SCT-1985, tales desplazamientos relativos variaron entre 0.5 y 2m, para amortiguamientos del AMR iguales a 30 y 5%, respectivamente.

Lo anterior hace ver que se deben desarrollar más estudios enfocados a establecer la forma de variación de la respuesta con el amortiguamiento del AMR, para diversos valores de las otras variables significativos (relaciones de masas, de períodos, y de amortiguamientos).

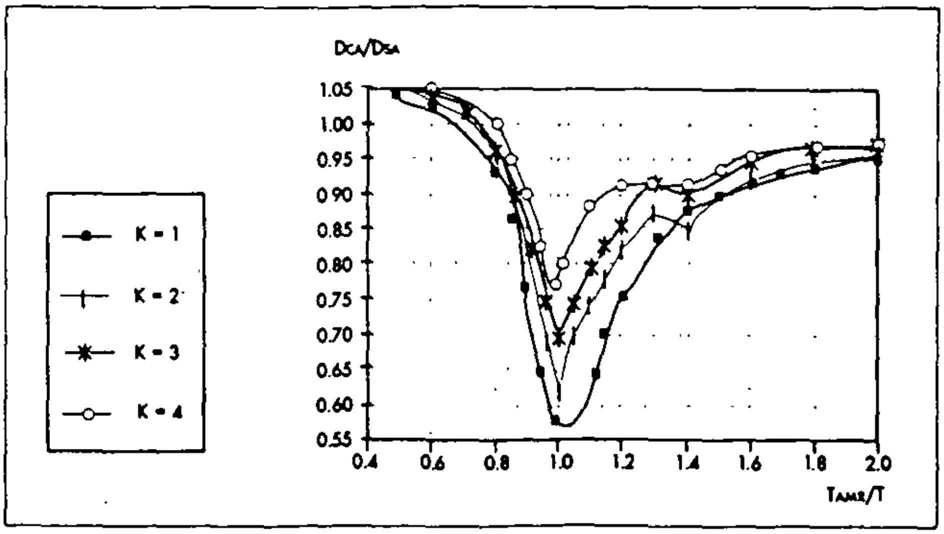
Influencia del comportamiento no lineal

En la literatura técnica sobre el tema existen estudios que demuestran que la respuesta sísmica de algunos edificios con comportamiento *lineal* se puede reducir muy eficientemente cuando se les incorpora un AMR en su azotea. Entre dichos estudios se encuentran varios en los que se supone como excitación el movimiento sísmico registrado en la SCT en 1985 Este registro se ha utilizado frecuentemente en los estudios, debido a que se trata de un sismo de larga duración y de banda estrecha, que cuenta con un número alto de oscilaciones con período de vibración casi constante. Por ejemplo, se ha analizado que un edificio de veinte niveles con un período fundamental de 2.03s sujeto al sismo SCT-1985 puede reducir su respuesta lineal hasta un 55% cuando se le incorpora un AMR en su azotea con relación de masa de 0.03, amortiguamiento de 5% del crítico y frecuencia igual a la del edificio. Esto significaría que para mantener comportamiento lineal dicho edificio se podría diseñar para soportar un coeficiente sísmico igual a 0.45, que es demasiado alto y daría lugar a secciones y armados excesivamente grandes en los miembros estructurales (Aguilar, Ruiz y Sánchez, 1993). Es decir, que aunque la respuesta se reduce de manera muy eficiente, en general la solución no es atractiva desde el punto de vista económico

Debido a lo anterior se han hecho estudios exploratorios sobre la eficiencia de los AMR considerando diferentes niveles de comportamiento no lineal de la estructura principal. Estos estudios se han realizado tanto con sistemas de dos grados de libertad como con marcos de múltiples niveles.

Los sistemas de dos grados de libertad tienen la limitación de no reflejar la influencia de los modos superiores de vibrar, ni los diferentes modos de falla; sin embargo, tienen la virtud de permitir evaluar, de manera gruesa, los efectos que tiene la inclusión de un AMR en la respuesta de sistemas no lineales, y a partir de esto, acotar el posible intervalo de validez del uso de este tipo de estructuración.

Figura 10 · Relación de valores medios de desplazamientos máximos de sistemas con AMR y sin éste.



Existen diferentes formas de analizar el fenómeno no lineal con un enfoque probabilista. Una es mediante el método de Monte Carlo, utilizando acelerogramas simulados y análisis paso a paso en el tiempo Otra alternativa consiste en linealizar las ecuaciones de movimiento y suponer la excitación como un proceso estocástico (estacionario o no estacionario). Esta última tiene la virtud de que ahorra tiempo de cómputo (con respecto al método de Monte Carlo); sin embargo, sus resultados son poco precisos para niveles elevados de no linealidad, debido, entre otras razones, a que deja de ser válida la hipótesis de que la respuesta puede considerarse como un proceso gaussiano.

* La influencia de la no linealidad de la estructura principal, cuando se le incorpora un AMR lineal, ha sido estudiada con los métodos antes mencionados (Ojeda y Ruiz, 1994; Ruiz y Esteva, 1996). Las tendencias generales son las siguientes:

1. A medida que la relación de períodos entre el AMR y la estructura principal (T_{AMR}/T) se aleja de la unidad, la efectividad del AMR es menor

2. Para relaciones de períodos T_{AMR}/T menores que 0.7 se presentan amplificaciones de la respuesta (en vez de presentarse reducciones de la respuesta) cuando se añade un AMR al sistema.

3. La efectividad del AMR es menor a medida que la no linealidad de la estructura principal crece Esto se puede ver en la figura 10 en donde se muestra el valor medio del desplazamiento del sistema con AMR entre el valor medio del desplazamiento del sistema principal para diferentes relaciones de períodos T_{AMR}/T, y para diferentes grados de no linealidad. K = 1 representa a un sistema lineal, y K = 4 a un sistema cuya fuerza de fluencia es igual a la resistencia demandada del sistema lineal dividida entre 4.

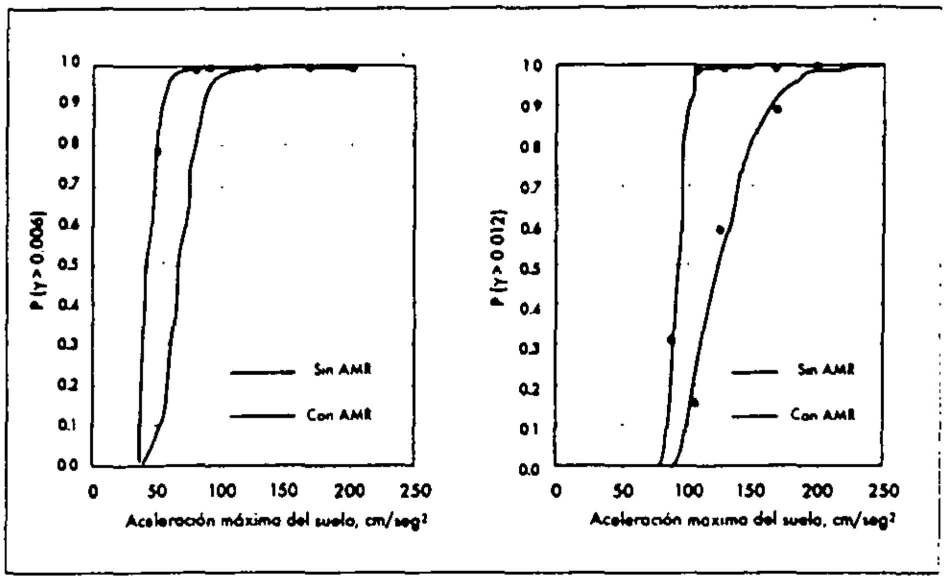


Figura 11 - Curvas de probabilidad de excedencia P(γ≥γ* |y) para el modelo diseñado con Q=2 a) Distorsión límite γ*=0.006, b)Distorsión límite γ*=0 012

De lo anterior se deduce que los AMR's se pueden emplear en estructuras diseñadas para desarrollar comportamiento no lineal ligero o moderado. Visto de otra manera, los AMR's son más eficientes ante temblores de intensidad pequeña o moderada que ante los de intensidad alta. Esto se comprobó en un estudio de sistemas equivalentes de dos grados de libertad sujetos a la acción de sismos con intensidades correspondientes a diferentes períodos de recurrencia (5, 20 25, 35, 50, 100 y 200 años). Se construyeron curvas de vulnerabilidad basadas en las probabilidades de alcanzar dos diferentes distosiones máximas de entrepiso (0.006 y 0.012), asociadas a los estados límite de servicio y de capacidad última, respectivamente. A partir de estas curvas y de la descripción del peligro sísmico del sitio, se calcularon las esperanzas de las tasas de excedencia de dichos estados límite (Soto Brito y Ruiz, 1998). En la figura 11 se muestran las probabilidades de excedencia para un sistema diseñado con un factor de comportamiento sismico Q = 2, y para dos distorsiones límite. En esta figura se puede ver que la diferencia de ordenadas entre la curva superior y la inferior es mayor para aceleraciones máximas del suelo per que para aceleraciones grandes. cir, que la eficiencia del AMR es mayor para sismos pequeños y moderados que para sismos grandes.

Comentarios finales sobre amortiguadores de masas resonantes

Los AMR son eficientes para controlar la respuesta de sistemas ante algunos tipos de solicitaciones, como por ejemplo el viento. Sin embargo, para el caso de excitaciones sísmicas, la mayor ventaja que ofrecen los AMR es su capacidad para contribuir a controlar la respuesta de las estructuras ante temblores correspondientes a períodos de recurrencia cortos y moderados, para los que es de esperarse que las respuestas de ambos sistemas se mantendrán en el intervalo de comportamiento lineal.

Al igual que para los dispositivos disipadores de energía, la decisión sobre su uso depende tanto de un análisis estru l, como de consideraciones sobre s totales y beneficios a largo plazo.

COMENTARIOS FINALES

Los estudios anteriores llevan a la conclusión de que para el control de la respuesta sísmica es más promisorio el uso de disipadores de energía, que el de amortiguadores de masas resonantes. La decisión sobre su empleo debe basarse en un análisis costo-beneficio.

Se considera que estas soluciones innovadoras no sustituirán a corto plazo a las tradicionales, pero cada día serán más aceptadas, a medida que se comprenda más su funcionamiento y que, además, su costo se reduzca (producción en serie, dispositivos más económicos, etc).

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MOUDLO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

REVISIÓN SOBRE EL DESARROLLO DE DISIPADORES DE ENERGIA SÍSMICA EN MEXICO

EXPOSITOR: DRA. SONIA E. RUIZ GOMEZ
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REVISIÓN SOBRE EL DESARROLLO DE DISIPADORES DE ENERGÍA SÍSMICA EN MÉXICO

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RESUMEN

Se hace una revisión bibliográfica sobre los estudios y aplicaciones prácticas realizadas en México por profesionales e investigadores en torno a disipadores de energía sísmica. Se tratan los temas siguientes: estudios experimentales, análisis en computadora, estudios de confiabilidad y optimación, criterios de diseño sísmico, uso de disipadores en la práctica, y reglamentos de diseño sísmico de edificios con disipadores. Al final se presentan algunos comentarios sobre estos temas y se proponen acciones que sería deseable realizar en el futuro.

INTRODUCCIÓN

En distintas partes del mundo se estudian nuevos materiales; en los laboratorios se prueban elementos y sistemas estructurales innovadores, se formulan nuevos criterios de diseño basados en el desempeño de las estructuras, tanto para el refuerzo de construcciones como para estructuras nuevas, y se proponen ideas para el control de la respuesta sísmica También existen grupos de trabajo orientados a escribir lineamientos de diseño aplicables a estructuraciones especiales. En algunos despachos de consultoría se realizan estudios comparativos sobre los costos totales de construcciones diseñadas convencionalmente, y los correspondientes a nuevas soluciones, como por ejemplo cuando se incluyen disipadores de energía sísmica.

México participa en esta evolución. Tanto investigadores como profesionales de la práctica se preocupan por comprender mejor el comportamiento estructural de sistemas diseñados en forma alternativa a la tradicional, así como por analizar la conveniencia de su uso desde el punto de vista económico, de seguridad y de comodidad para los ocupantes de los inmuebles.

Prueba de esta participación son los trabajos presentados en el simposio "*Disipadores de energía para controlar la respuesta sísmica de edificios*", organizado por la Sociedad Mexicana de Ingeniería Sísmica. En este simposio se han reunido profesionales interesados en el tema para discutir e intercambiar ideas sobre análisis y estudios desarrollados en México en esta dirección. Además de los trabajos publicados en las memorias de este simposio, existen en nuestro país otros desarrollos valiosos que contribuyen a nuestro acervo de conocimientos en esta materia. Algunos de estos trabajos se mencionan en las referencias al final de este documento; otros se escapan de nuestro conocimiento Se pide a los autores de dichos trabajos perdonar estas omisiones involuntarias.

ANTECEDENTES

El tema sobre el **control pasivo** de la respuesta sísmica de edificios se ha estudiado en México desde hace algunos años. No así el tema de **control activo,** que permanece inexplorado en nuestro país La razón de esto se debe en parte a que el tema de control activo es mucho más complejo que el pasivo, tanto desde el punto de vista matemático como de sus implicaciones prácticas (mantenimiento, fuente de poder auxiliar, costo más elevado que los disipadores de control pasivo, análisis estructural más complejo, fuerzas de control muy grandes con altas velocidades de reacción, algoritmos de control confiables, niveles de tecnología altos, etc).

Durante los últimos años, se han hecho en México análisis experimentales en elementos y en sistemas estructurales, así como análisis de sistemas estructurales en computadora, empleando modelos y probabilistas; estos últimos, en estudios relacionados con confiabilidad estructural ante sismos. Dichos estudios comprenden tanto análisis paramétricos de sistemas de un grado de libertad, como análisis de modelos más refinados de edificios altos. Los análsis han cubierto tanto los problemas de respuesta estructural como los de costos de construcción. Asimismo, recientemente se han formulado criterios de diseño de estructuras con estos dispositivos. En la práctica profesional, se han instalado disipadores de energía en algunos edificios de la ciudad de México.

Esta revisión de la literatura se refiere a los estudios y aplicaciones desarrollados en México, en el área de sistemas disipadores de energía sísmica. Los conceptos cubiertos se agrupan como sigue:

- Estudios experimentales
- Análisis de edificios y de sistemas en computadora
- Estudios sobre confiabilidad y optimación
- Uso de disipadores de energía en la práctica
- Criterios de diseño sísmico
- Reglamentos de diseño sísmico

ESTUDIOS EXPERIMENTALES

Los primeros elementos disipadores que se probaron en México fueron las soleras de acero en forma de U. La idea sobre utilizar esta forma como disipador sísmico fue publicado por primera vez por Kelly *et al* en 1972. Esta misma idea se propuso en México originalmente para controlar el hundimientos de cimentaciones hechas con pilotes de fricción (Aguirre y Chicurel, 1976), Posteriormente la idea evolucionó para aplicarla a disipadores de energía sísmica. Hasta la fecha este tipo de disipadores no se han instalado en estructuras.

Las pruebas experimentales de disipadores de solera en forma de U han sido conducid
básicamente por M. Aguirre y R. Sánchez en el Instituto de Ingeniería de la UNAM (I. I. - UNA

Aguirre et al, 1976, 1989). Se ha hecho un número de pruebas de dispositivos hechos con acero y con aluminio. Se han producido curvas de fatiga que relacionan el número de ciclos que resiste el dispsitivo con la amplitud del movimiento

Con el fin de optimizar el espacio ocupado por los elementos en forma de U, se propuso una forma de colocar varios disipadores en un solo conjunto (Aguirre, 1993). Esta propuesta se probó en el Centro Nacional de Prevención de Desastres (CENAPRED) en un marco de acero ante una carga lateral alternada dada por un actuador horizontal (Echavarria *et al*, 1996). Usando el mismo sistema de apoyo (pared de reacción, actuador y marco) se probaron en la misma institución otros disipadores basados en la fluencia del acero. Las caracteristicas sobre el comportamiento de dichos dispositivos ante cargas alternadas se detallan en este volumen.

Los disipadores en forma de U también se probaron en una estructura de acero de dos niveles sobre la antigua mesa vibradora del I. I. - UNAM (González Alcorta *et al*, 1985, 1994). De estos ensayes se aprendió sobre muchos detalles de tipo constructivo, relacionados sobre todo con la vibración local de los elementos diagonales. En las memorias de este simposio se publica un articulo sobre los resultados de estas pruebas.

Posteriormente, Ortega (1998) propuso disipar energia mediante placas rectangulares trabajando como vigas sujetas a cargas concentradas aplicadas simétricamente a ambos lados del centro de su claro. A principios de 1998 Ortega solicitó al I. de I. -UNAM probar en su laboratorio una serie de placas de acero con distintas dimensiones ante desplazamientos armónicos (Escobar *et al*, 1998). Algunos resultados de dichas pruebas se presentan en esta memoria.

Hasta donde se tiene conocimiento, los ensayes mencionados en los párrafos anteriores son todos los que se han realizado en México sobre disipadores de energía sismica. Es obvio que el número de ensayes son muy pocos comparados con los que se realizan en otros países con alto riesgo sismico, como Japón, Estados Unidos de Norteamérica (EEUU), Italia y Canadá. También son pocos en comparación con el número de estudios sobre modelos numéricos que se han realizado en nuestras instituciones sobre este tema

Los disipadores tipo ADAS, como los instalados en varios edificios de la ciudad de México, han sido probados en el laboratorio de la Universidad de California en Berkely (Alonso, 1990).

En México se han propuesto otros tipos de disipadores, como el panel propuesto por Vera y Ramírez de Alba (1996), sobre el que se han hecho algunos estudios analíticos, y los disipadores imantados descritos de manera general por Echegaray (1997). Sin embargo, ninguno de estos dos se ha probado en laboratorio.

ANÁLISIS DE EDIFICIOS EN COMPUTADORA

El estudio analítico de edificios con disipadores de energía en nuestro país inicia desde hace aproximadamente diez años (Ocampo *et al*, 1987; Arboleda, 1988; Chávez Gómez y González Alcorta, 1989). Desde entonces se ha hecho un gran número de análisis numéricos en computadora. Estos comprenden tanto marcos de un nivel y una crujia, como edificios en tres dimensiones de múltiples niveles y varias crujias

E Rosenblueth plantea un programa de trabajo sobre análisis numéricos de sistemas con disipadores que desafortunadamente no logra ver realizado.

Análisis de Estructuras de Múltiples Niveles

La mayoria de los análisis realizados en México tratan con disipadores de tipo histerético (ADAS. TADAS y soleras de acero). Algunos de los trabajos relativos a estructuras de varios niveles con disipadores de energía son los que siguen:

Vargas et al (1991) analizan un marco de concreto reforzado de nueve niveles y tres crujias, con periodo inicial de vibración de 0.83 s. Este se supone ubicado en la zona blanda del Distrito Federal Los autores comparan el comportamiento del marco suponiendo dos métodos alternativos de rigidización con muros de concreto y con dispositivos disipadores. El periodo final de la estructura con disipadores es de 0 62s. Los autores hacen ver que en el primer caso (muros de concreto) la solución es buena, pero que la fuerza cortante basal es muy grande; concluyen que también los disipadores son una buena solución. No se presentan análisis de costos.

Córdova y González Alcorta (1992) realizan un estudio más completo que el anterior. Se trata de un edificio existente, localizado en la zona de transición de la ciudad de México El periodo dominante de la excitación es de 1.15s. El edificio es de nueve niveles y tres crujias, con periodo en la dirección transversal (medido con vibración ambiental) igual a 1.14s. Los autores suponen cuatro sistemas de rigidización muros de mamposteria, muros de concreto, diagonales de acero y una combinación de diagonales de acero con disipadores de energía. El periodo de la estructura co diagonales de acero y disipadores de energía es de 0.56s Los autores concluyeron que la mejo alternativa de rigidización es a base de muros de concreto. Concluyen también que los disipadores de energia propuestos para este caso no funcionaron adecuadamente Aclaran que si se supusieran otros niveles de fluencia de dichos elementos, probablemente sí habrian disipado energia eficientemente

Valles Mattox (1993) analiza un edificio hipotético de dieciseis niveles con periodo de vibración igual a 2.31s, que es mayor que el periodo dominante de la excitación (igual a 1.2s) El edificio reforzado con contravientos tiene un periodo de 1.13s, mientras que para el que se refuerza con disipadores se estima un periodo de 1.48s El autor hace ver las ventajas (en cuanto a costos y a respuesta estructural) que presenta el refuerzo con disipadores en comparación con empleo de contravientos, en estructuras con periodos comprendidos en la zona descendente el espectro de aceleraciones Para el caso en estudio se concluye que las fuerzas sobre la cimentación son muy altas, aún utilizando disipadores, por lo que recomienda reducir el número de pisos. El autor opina que los mayores ahorros que se tienen al utilizar disipadores de energia se relacionan con el refuerzo de la cimentación y no con la superestructura. También expresa que los costos de construcción en estructuras nuevas con disipadores son similares a los que resultan de emplear sistemas tradicionales.

Tena, Gómez y Vargas (1993) analizan la respuesta de dos edificios de doce niveles reforzados con disipadores de energía. Sus periodos de vibración son de 1.5s y 1.2s. El periodo del terreno en el sitio de desplante es de 0.90s Estas estructuras no tuvieron daños durante el temblor de 1985; embargo, de acuerdo con las especificaciones del reglamento vigente deben ser reforzadas, ya qu su capacidad sismo-resistente es menor que la requerida para estructuras tipo A ubicadas en la zona II. Los autores concluyen que el refuerzo propuesto a base de disipadores mejora sustancialmente la ductilidad y la capacidad sismo-resistente de los inmuebles En el trabajo no se analizan soluciones de refuerzo alternativas.

Urrego, Ruiz y Silva (1993) analizan la respuesta de un marco de diez niveles con disipadores colocados según cuatro distintos arreglos espaciales. El periodo de vibración del marco sin disipadores es de 1s. Se supone ubicado en la zona de terreno blando del Distrito Federal (SCT). El objetivo del estudio es determinar cual de los arreglos propuestos es el más adecuado. Dicho análisis se hace desde el punto de vista de respuesta estructural y de economía. Los autores hacen ver que los arreglos más convenientes son aquéllos en los cuales las diagonales están dispuestas de manera de evitar la ocurrencia de grandes fuerzas axiales sobre las columnas que transmiten los momentos de volteo a la cimentación.

En un estudio posterior (Urrego y Ruiz, 1994) se demuestra que, para el mismo caso del párrafo anterior, las fuerzas máximas en columnas y los momentos máximos de volteo son 15% menores a los correspondientes a un marco contraventeado con similar periodo de vibración. Por otro lado, los desplazamientos máximos del marco con disipadores son 33% mayores que los del contraventeado Se obtiene que el costo de construcción del edificio con disipadores es 3.5% mayor que un edificio similar (con el mismo periodo de vibración) diseñado convencionalmente, de manera que presente el mismo nivel de daño (Ruiz et al, 1995).

Vargas *et al* (1993a, 1993b) presentan un estudio comparativo sobre respuestas dinámicas de una torre de acero de diez niveles y un edificio de concreto reforzado de nueve niveles. Modifican sus características dinámicas de modo que sus periodos varíen entre 1.0 y 2.5s. Se analizan diferentes alternativas de refuerzo, entre las que se encuentran los disipadores de energía. Se utiliza como excitación el registro obtenido en la SCT en 1985 (SCT-85). Los autores hacen ver que una de las ventajas de la torre de acero es que permite desplazamientos relativos de entrepiso mayores que los que permite el edificio de concreto antes de que se presente la fluencia de alguno de sus elementos. Esto hace que los disipadores entren en funciones y mejoren la respuesta de la estructura antes de que presente daños. Aunque los modelos con disipadores de energía muestran una importante disminución de las solicitaciones a las que se somete la estructura original, no siempre se logra que la disipación de energía se concentre totalmente en dichos dispositivos. Para estos casos no es evidente que los sistemas de disipación de energía sean la mejor elección para reducir la respuesta sísmica de las estructuras.

Ávila y Guitérrez (1994) estudian la respuesta dinámica no lineal de un edificio de tres niveles tipo escuela, con disipadores de energía y sin ellos El periodo de esta escuela según el diseño convencional, es de 0.75s El diseño se realiza usando dos diferentes factores de comportamiento sísmico: Q = 2 y Q = 4 La estructura se supone ubicada en la zona blanda de la ciudad de México. Se supone como excitación el acelerograma registrado en la SCT en 1985. Para el diseño del edificio con disipadores se considera que estos toman el 50% de la fuerza cortante de entrepiso. Los autores concluyen que la inclusión de los disipadores de energía mejora el comportamiento de la estructura. Estas mismas conclusiones obtienen estos autores al analizar un edificio nuevo de diez niveles ubicado en un sitio cercano a la SCT. El periodo del edifico sin disipadores en este caso es de 1.46s (Ávila y Gutiérrez, 1996).

Ruiz, et al (1996) analizan los costos de un edificio nuevo de veinte niveles diseñado convencionalmente y otro con disipadores de energia Los autores calculan que el costo de construcción del edificio con disipadores es 9.5% veces más grande que el edificio convencional (con igual periodo de vibración) y con el mismo nivel de daño cuando se le somete a la acción del sismo SCT-1985.

Jara (1996) estudia el comportamiento de dos edificios, uno de 6 niveles (con periodo natural de vibración de 1 17s), y otro de 15 niveles (con periodo de vibración de 1 95s). Estos se someten a la acción del acelerograma SCT-85. Se plantean dos alternativas como refuerzo: usar contravientos o disipadores ADAS Los resultados hacen ver que los disipadores colocados en el edificio de seis niveles son muy eficientes en reducir la respuesta, sin incrementar las fuerzas cortantes totales. Esto no sucede en el edificio de quince niveles. En este caso el uso de disipadores de energia es muy poco eficiente y conviene más utilizar contravientos como refuerzo.

Limón y Ruiz (1997) estudian un marco estructural con periodo de vibración de 2s que debe reforzarse debido a su cambio de uso. En el estudio se suponen tres distintas distribuciones espaciales de disipadores. El edificio se somete a la acción del movimiento registrado en la estación SCT en 1985. Los autores demuestran que, para ese caso, es mejor reforzar el edificio con contravientos que con disipadores de energia sismica Esta recomendación se basa tanto en un análisis de la respuesta estructural como en un análisis de costos de construcción.

Tena y Vergara (1997) analizan dos alternativas de refuerzo de un edificio existente de diez niveles con periodo de vibración original de 1.96s, ubicado cerca del parque de la Alameda en el centro de la ciudad de México. Se hace la observación de que las ordenadas espectrales de seudoaceleració del registro empleado (N-S Alameda-85) para 2% de amortiguamiento, son similares tanto para lo periodos de la estructura reforzada con contravientos (0.9s) como con disipadores ADAS (1.19s). Debido a lo anterior, las fuerzas máximas cortantes de entrepiso resultan mayores para el caso en que se usan contravientos que para el caso en donde se usan disipadores, debido a que la estructura con disipadores presenta mayor amortiguamiento Asimismo, las fuerzas transmitidas a la cimentación son mayores para el caso de refuerzo con contravientos. Los desplazamientos de entrepiso son similares para ambas alternativas de refuerzo. Por otro lado, para el caso en estudio, los costos iniciales del refuerzo de la superestructura con disipadores ADAS resultan 1.91 veces mayores que los que utilizan contravientos Se concluye que es mejor utilizar contravientos debido a que en este caso la cimentación no requiere reforzarse. Probablemente si requiriera reforzarse convendria más utilizar disipadores ya que estos dan lugar a menores solicitaciones en la cimentación.

Análisis Paramétricos de Sistemas de un Nivel

En México se han realizado estudios para ver la influencia de distintos parámetros en la reducción de la respuesta de sistemas con disipadores de energia tipo histerético. Algunos de estos estudios son los siguientes.

Gómez, Rosenblueth y Jara (1993) analizan marcos de nivel y una crujia ante tres sismos tipicos del valle de México, correspondientes a terrenos blando, intermedio y duro Varian el periodo fundamental original, la rigidez final de los modelos y el desplazamiento de fluencia de dispositivos ADAS Los autores muestran que al incluir dispositivos disipadores o diagonales acero se reduce la deformación máxima del sistema original. Se compara el comportamiento de modelos marco-disipadores y el de los modelos rigidizados con contravientos. Los autores concluyen que cuando los modelos originales se refuerzan de manera que su rigidez final es 2.5 veces la del marco original, resulta más adecuado utilizar contravientos que disipadores de energia, siempre y cuando el periodo se encuentre entre 1 y 2s. Se indica que para estructuras que tengan periodos iguales o mayores que 2s puede ser mejor utilizar disipadores que contravientos, cuando estas estructuras se ubican en terrenos como el de la SCT, el cual tiene un periodo dominante de 2s Se subraya que en algunos casos los refuerzos pueden incrementar el costo de la cimentación por aumento en los momentos de volteo. Los autores indican que se requieren estudios costo-beneficio para definir las condiciones de aplicabilidad práctica de los dispositivos.

Arista y Gómez (1993) presentan un estudio paramétrico de sistemas estructurales de un nivel con disipadores de energia y con rigidez asimétrica en planta. Dichos sistemas se someten a la acción de registros sísmicos correspondientes a zonas de terrenos duro, intermedio y blando de la ciudad de México Se supone que la asimetria que se trata en este caso es producto de una mala instalación de los disipadores, de defectos en la construcción de los mismos o de un mal programa de mantenimiento. Los autores recomiendan que los criterios de diseño de estructuras con disipadores deben tomar en cuenta las posibles excentricidades de rigidez causadas por los dispositivos, ya que estos pueden causar grandes concentraciones de ductilidad en algunos elementos estructurales

Jara (1998) elabora espectros de ductilidad y de distorsión angular de sistemas de un grado de libertad ante el registro SCT-85, para dos resistencias del sistema original. En dicho estudio se analizan estructuras reforzadas con contravientos, y reforzadas con disipadores. Se supone que ambas tienen el mismo periodo inicial. No se establece la condición de que el daño sea el mismo en ambas estructuras. Debido a las suposiciones antes mencionadas, el autor llega a la conclusión de que en la zona descendente del espectro de SCT no es recomendable utilizar disipadores de energia para el refuerzo de edificios, y que en la zona comprendida entre 1 y 2 s es más recomendable utilizar disipadores en vez de contravientos. Estas conclusiones son muy diferentes a las publicadas por Martínez Romero (1993), Valles Mattox (1993), Gómez *et al* (1993), Jara (1996), y Limón y Ruiz (1997).

Alvarez y Ruiz (1997) realizan un estudio sobre la influencia que tiene la rigidez flexionante de las trabes con respecto a la de las columnas en el desempeño de los disipadores TADAS y ADAS. Asimismo, analizan la influencia de la modelación estructural de los disipadores en la respuesta de sistemas de un nivel y una crujia. Por otro lado, mediante el método del elemento finito, evalúan la influencia de la carga axial en ambos los disipadores ADAS. Concluyen que, para ciertos casos, se pueden generar cargas axiales sobre las placas de los disipadores, lo que modifica su capacidad de deformación. Esto generalmente no se considera en el diseño.

Programas de cómputo

Los programas de cómputo más utilizados en México para el análisis de estructuras con disipadores de energia son los siguientes ETABS-6, SAD-SAP, DRAIN-TABS, DRAIN-2D y DRAIN-2DX. En las últimas fechas se ha empezado a utilizar el programa SAP-2000.

ESTUDIOS SOBRE CONFIABILIDAD Y OPTIMACIÓN DE SISTEMAS CON DISIPADORES

En los últimos años se han realizado avances importantes en los planteamientos de confiabilidad de sistemas con disipadores. Enseguida se mencionan las principales aportaciones en esta línea de investigación

Esteva y Díaz (1993a, 1993b) han formulado modelos matemáticos que describen la acumulación de daño, la probabilidad de transición entre estados sucesivos de un sistema y la evolución de la confiabilidad del sistema con la historia sísmica. Dentro de los conceptos sobre daño acumulado se involucra el desarrollo de modelos constitutivos del comportamiento de elementos estructurales con degradación de rigidez y de resistencia (Rodríguez y Esteva, 1993; Díaz y Esteva, 1993).
Usando estos conceptos se han realizado estudios sobre la influencia de la ductilidad de diseño y la distribución espacial de los disipadores de energía sobre las demandas de ductilidad e indices de daños (Campos, 1998).

Esteva, Díaz y García (1998) han establecido criterios para diseño, reparación y mantenimiento de sistemas con disipadores basados en la minimización de la suma de los costos iniciales, de daño y de mantenimiento

USO DE DISIPADORES DE ENERGÍA EN LA PRÁCTICA

Los primeros disipadores de energía que se utilizaron en México fueron los tipo ADAS (Martíne. Romero, 1993). Disipadores de este tipo se han colocado en tres edificios de la ciudad de México y en dos de Acapulco, Gro. A continuación se hacen algunos comentarios sobre cada uno de estas estructuras

El primer edificio reforzado en México con disipadores (ADAS) fue el del Hospital de Cardiologia, ubicado dentro del centro Siglo XXI del Instituto Mexicano del Seguro Social (IMSS). Se trata de un edificio de cinco niveles sobre el nivel del terreno. El refuerzo en este caso se hizo de forma externa, de modo que la obra no interfiriera con la operación normal del edificio, lo que significó una gran ventaja.

El segundo edificio reforzado con disipadores ADAS es uno de doce niveles, localizado en Izazaga 38. Sus periodos fundamentales en los sentidos longitudinal y transversal eran de 2.33s y 3.82s antes de reforzarse, y de 2.01s y 2.24s respectivamente, después de reforzarse con disipadores. El responsable de estos trabajos (Martínez Romero, 1993) opina que si este edificio se hubiese reforzado en forma convencional se habrían incrementado sustancialmente tanto las cargas cortantes basales como las cargas actuantes sobre la cimentación.

Actualmente se está finalizando el refuerzo de un tercer edificio con disipadores ADAS. Este corresponde a las oficinas centrales del IMSS. Está ubicado en Av. Reforma 476. Se trata de un edificio de once niveles más planta baja y sótano localizado en la zona de transición (II) de la ciudad de México, con periodo dominante de vibración de 1.2s. En el cuerpo central del edificio en uno de los laterales se han instalado siete acelerómetros digitales distribuidos en la azotea, en nivel 3 y en el sótano (Pérez Rocha *et al*, 1997). El edificio estaba totalmente reforzado hasta el nivel 3 en noviembre de 1997. Hasta esta fecha los acelerómetros habian registrado los efectos de seis temblores, dos de ellos con magnitud 7.3 (el de Ometepec y el de Playa Azul) Ante el movimiento provocado por de uno de estos eventos (Ometepec, 14 septiembre de 1995) se identificó que los disipadores incursionaron en el intervalo no lineal (hasta esa fecha se habían reforzado la planta baja, el mezzanine, el nivel 1 y 60% del nivel 2).

Existen otras estructuras (nuevas) construidas con disipadores ADAS en Acapulco Gro Estas son la terminal portuaria TMM (Trans Marit. Mex.) y dos torres altas de condominios (Juárez, 1998).

En México se ha utilizado también otro tipo de disipadores para el refuerzo de un edificio. Estos son los que disipan energía por fricción (Sánchez Martinez, 1993); se emplearon en el sistema de diagonales de acero que constituye el refuerzo del Hospital 20 de Noviembre del ISSTE, en la esquina de las Avs Coyoacán y Félix Cuevas en el Distrito Federal.

Recientemente se construyó un edificio de cinco niveles con disipadores viscoelásticos (Miranda *et al*, 1998). Este edificio alberga una sucursal de la compañía norteamericana 3M, misma que produce los disipadores que se incluyeron en el edificio. Este se ubica en el fraccionamiento Santa Fé, al oeste de la ciudad de México. El terreno en esa zona es duro. El edificio tiene un periodo de vibración cercano a 1s. Los responsables del análisis estructural (Miranda *et al*, 1998) hacen notar que este tipo de dispositivos disipan energía aún para niveles bajos de deformación. Para el diseño de la estructura (basado en el desempeño de la estructura) se tomó en cuenta que los disipadores viscoelásticos son sensibles a los cambios de temperatura.

CRITERIOS DE DISEÑO SÍSMICO DE EDIFICIOS CON DISIPADORES

En México, como en todo el mundo, existen pocas recomendaciones normativas sobre el diseño sísmico de edificios con disipadores. Algunas propuestas de autores mexicanos son las siguientes:

Silva y Ruiz (1993) proponen un procedimiento iterativo para el diseño de marcos estructurales nuevos con disipadores de energía de tipo histerético. Debido a las condiciones particulares del estudio en dicho trabajo se impone que el marco con disipadores tenga un periodo de vibración igual al marco convencional del cual se parte (condición que no debe imponerse en general). Con el fin de comparar resultados, además se pone como condición que el daño en ambas estructuras sea similar. Las condiciones de diseño que se establecen son las siguientes: 1) que la rigidez de los disipadores varíe de manera proporcional a la rigidez de los entrepisos, 2) que estos tengan suficiente capacidad de deformación para que los disipadores trabajen adecuadamente, 3) que la ductilidad desarrollada por los disipadores sea adecuada, 4) que el marco que contiene a los disipadores tenga un comportamiento casi lineal, y 5) que las distorsiones de entrepiso no sean mayores a las recomendadas por el Reglamento de Construcciones del Distrito Federal. Este procedimiento implica realizar varios análisis no lineales paso a paso en el tiempo.

Sánchez *et al* (1993) plantea diferentes alternativas para instalar disipadores en marcos estructurales y algunas bases muy generales para el diseño de estructuras con estos dispositivos. Además, proponen un criterio para evaluar en forma empírica el grado de daño acumulado en los disipadores durante su vida útil.

Esteva y Veras (1998) y Esteva *et al* (1998) proponen un criterio para el diseño sísmico de edificios nuevos y para el refuerzo de edificios con disipadores de energía. Las condiciones de diseño se expresan en términos de las ductilidades nominales de disipadores y estructura, de los valores permisibles de estas ductilidades, y del desplazamiento relativo de entrepiso. El temblor de diseño se expresa en términos de espectros no lineales de aceleraciones para distintas ductilidades demandadas. Para llegar al diseño final es necesario realizar iteraciones para lograr que todas las condiciones de diseño se satisfagan. Los pasos detallados de este proceso de diseño se encuentran en un artículo de estas memorias.

REGLAMENTACIÓN SOBRE DISIPADORES EN MÉXICO

El concepto de disipación externo de energía sísmica es relativamente nuevo dentro del diseño de edificios. La reglamentación oficial en todo el mundo es mínima. En México aún no se cuenta con una normatividad oficial para diseño de este tipo de estructuración.

Ruiz y Alvarez (1995) revisaron la normatividad oficial de treinta y siete países, miembros de la Asociación Internacional de Ingeniería Sísmica. Se puso especial énfasis en el diseño, construcción y refuerzo de edificios con elementos reductores de la respuesta sísmica (aisladores de base y disipadores de energía sísmica). Se encontró que la mayoría de las normas revisadas permiten utilizar estos dispositivos con la condición de que los diseños sean aprobados por las autoridades correspondientes. También es el caso del Reglamento de Construcciones del Distrito Federal.

El Comité de Normatividad y Criterios de Diseño Sísmico de la Sociedad Mexicana de Ingeniería Sísmica, coordinado por el Dr L. Esteva, presentará dentro del programa del Simposio, algunas ideas sobre la reglamentación sobre edificios con disipadores de energía sísmica. Se espera que durante la sesión correspondiente participen los asistentes al evento, proponiendo al Comité algunas acciones a seguir.

COMENTARIOS FINALES

A continuación se hacen algunos comentarios sobre los temas revisados:

1. Los pocos estudios experimentales hechos en nuestro país se han realizado sólo en dos instituciones: el I. de I - UNAM y el CENAPRED, ambos ubicados en la ciudad de México. No se han realizado pruebas en instituciones del interior de la República. Esto en parte se debe a la falta de instalaciones de tipo experimental, y en parte a la falta de interés por este tipo de estudios, quizá porque normalmente consumen gran cantidad de tiempo, de energía y de recursos económicos

2. Gran parte de los análisis realizados en México tratan con disipadores basados en la deformación plástica del acero. Se le ha dado mucho menor importancia al estudio de disipadores compuestos por materiales viscosos o viscoelásticos. Es deseable que en el futuro se cubrieran también estos temas, incluyendo estudios de tipo económico.

3 Los esfuerzos dedicados a estudios analíticos en computadora son mucho mayores que los invertidos en pruebas experimentales y en análisis costo-beneficio Es necesario dedicar mas esfuerzos a estas dos lineas.

4 También es necesario dedicar esfuerzos a establecer lineamientos de diseño de edificios con disipadores de energia El Comité de Normatividad y Criterios de Diseño Sismico de la SMIS podria dar recomendaciones para coordinar dichos esfuerzos

5. Hasta la fecha ha habido poca discusión e intercambio de experiencias entre los distintos grupos de trabajo (entiéndase distintas instituciones) dedicados al estudio de este tema.

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XXVI CURSO INTERNACIONAL DE INGENIERIA SÍSMICA

MOUDLO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

PERFORMANCE-BASED DESING APPROACH FOR SEISMIC REHABILITATION OF BUILDING UIT DISPLACEMENT – DEPENDENT DISSIPARORS

EXPOSITOR: DRA. SONIA E. RUIZ GOMEZ
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EDIFICIOS EN LOS QUE CONVIENE REFORZAR CON DISIPADORES DE ENERGÍA EN LUGAR DE UTILIZAR CONTRAVIENTOS.

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RESUMEN

En este trabajo se realiza un análisis dinámico no lineal de tres marcos convencionales ubicados en el suelo blando de la Cd de México de 10, 20 y 33 niveles, con periodos iniciales de 1s, 2s y 3s respectivamente, reforzados con dos alternativas diferentes: disipadores de energía y elementos de contravientos. Se compararan sus respuestas y se proporcionan algunas recomendaciones para determinar en qué casos es más conveniente reforzar con disipadores y en cuales con contravientos.

SUMMARY

A dynamic analysis of three frames reinforced in accordance with two different alternatives is presented. The solutions considered are energy dissipating systems and bracing elements. The frames have linear fundamental periods of 1, 2 and 3 s Their seismic responses are compared. Some recommendations are proposed regarding the conditions that make either of the solutions more convenient than the other

INTRODUCCIÓN

El uso de sistemas disipadores de energía para reforzar edificios sujetos a sismos intensos cada vez es más frecuente, pero en México, como en muchos otros países con alto riesgo sísmico, no existen recomendaciones oficiales para diseño de edificios nuevos con disipadores de energía, ni para reforzar edificios existentes con dichos dispositivos (Ruiz y Alvarez, 1996). Este tipo de dispositivos, en muchas ocasiones, permite reducciones apreciables en la respuesta sísmica, lo que permite aminorar o eliminar la posibilidad de daños en la estructura. El concepto básico es que la energía se disipe a través del trabajo que desarrollen estos dispositivos y no por el comportamiento dúctil de los elementos de la estructura. Por esta razón se hacen esfuerzos orientados a conocer y proponer diferentes tipos de sistemas disipadores, a estudiar su influencia en la respuesta dinámica de los sistemas estructurales que los contienen, así como a desarrollar criterios y métodos aplicables a la práctica del diseño, tanto para el diseño de nuevas estructuras como para el refuerzo de edificios existentes.

El Reglamento de Construcciones para el Distrito Federal (RCDF-1993) permite utilizar estos dispositivos con la condición de que los diseños conduzcan a una confiabilidad por lo menos igual a la que se obtendría para estructuras convencionales empleando los métodos y requisitos especificados en el propio reglamento Los diseños finales deben ser aprobados por las autoridades correspondientes para poder otorgar el permiso de construcción

El objetivo de este trabajo esta orientado a conocer más adecuadamente el uso eficiente de los disipadores. Para ello se analizan las respuestas estructurales de tres edificios de 10, 20 y 33 niveles reforzados con disipadores y con contravientos. Se suponen ubicados en el suelo blando de la Cd. de México. Se comparan sus respuestas y se recomienda para cuales edificios es más conveniente el uso de disipadores en lugar de utilizar contravientos como refuerzo

DESCRIPCIÓN DE LOS EDIFICIOS

Con la idea de realizar un estudio en donde se abarquen las principales zonas del espectro de respuesta del sismo registrado en la Secretaria de Comunicaciones y Transporte del 19 de septiembre de 1985, se modelaron tres marcos de 10, 20 y 33 niveles de concreto reforzado con un periodo fundamental To de 1, 2 y 3 s respectivamente. Se diseñan para uso de oficinas. Se clasifican como estructuras del grupo B. Se ubican en la zona de lago del Valle de México considerada como zona III tomando en cuenta las disposiciones de seguridad estructural para el Reglamento de Construcción del Distrito Federal de 1976 (RCDF76) En la Figura 1 se presentan las principales características geométricas para cada uno de los edificios, y en la Tabla 1 las secciones de las trabes y columnas.

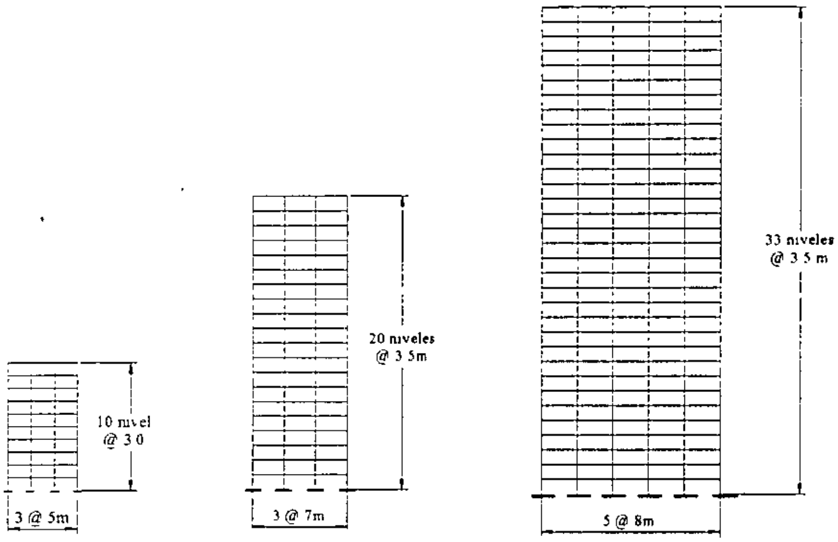


FIGURA 1 Elevación de los marcos sin reforzar (SR).

TABLA 1. Secciones de trabes y columnas de los marcos.

MARCO DE 10 NIVELES		
Nivel	**Columnas (cm)**	**Trabes (cm)**
1-4	56 x 56	75 x 35
5-6	54 x 54	
7-8	50 x 50	
9-10	42 x42	

MARCO DE 20 NIVELES		
Nivel	**Columnas (cm)**	**Trabes (cm)**
1-4	120 x 120	95 x 45
5-8	110 x 110	
9-12	100 x 100	
13-16	90 x 90	
17-20	80 x 80	

MARCO DE 33 NIVELES	
Nivel	**Columnas (cm)**
1-5	190 x 190
6-10	170 x 170
11-15	150 x 150
16-21	130 x 130
22-25	120 x 120
26-30	110 x 110
31-33	90 x 90

Nivel	Trabes (cm)
1-12	100 x 90
13-27	100 x 80
28-33	90 x 70

DESCRIPCIÓN DEL MOVIMIENTO SÍSMICO

Para el estudio de los marcos se utilizó un acelerograma representativo de una zona blanda del Valle de México. Este corresponde al componente este-oeste del movimiento registrado en la Secretaria de Comunicaciones y Transporte durante el sismo del 19 de septiembre de 1985 (SCTEWRE85). En la Figura 2 se muestra su espectro de respuesta lineal.

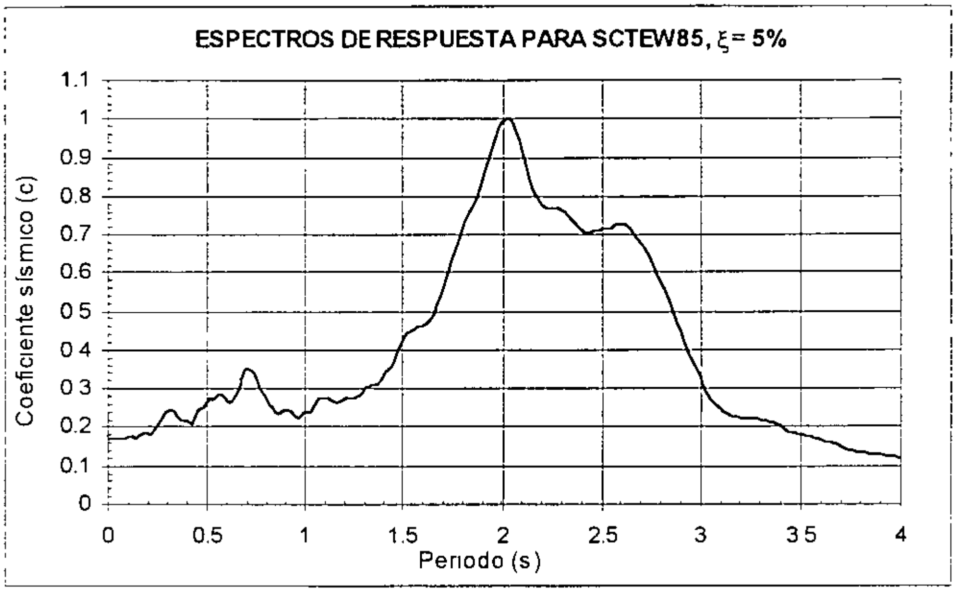
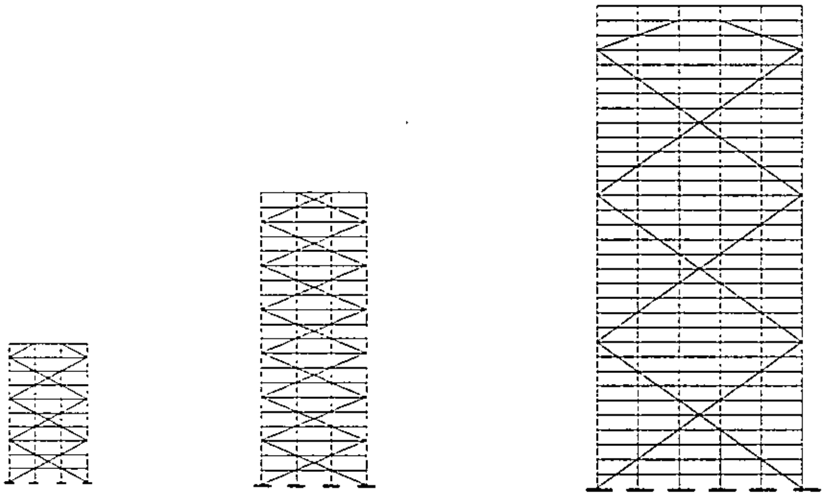


FIGURA 2. Espectro de respuesta lineal para SCTEW85.

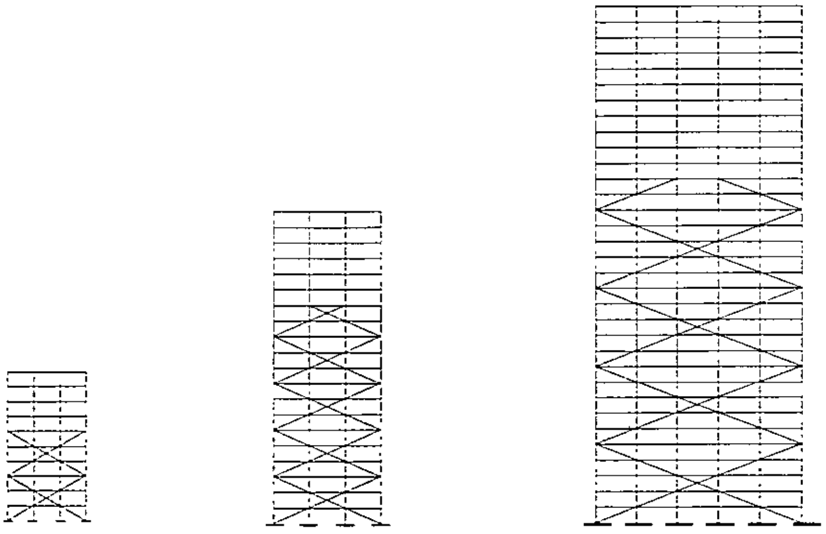
ARREGLO DE LOS EDIFICIOS REFORZADOS CON DISIPADORES Y CONTRAVIENTOS

En las Figuras 3 y 4 se muestran los arreglos finales de los marcos de 10, 20 y 33 niveles reforzados con contravientos y disipadores respectivamente.



a) Marco de 10 niveles b) Marco de 20 niveles c) Marco de 33 niveles

FIGURA 3. Arreglo de los contravientos en los marcos reforzados



a) Marco de 10 niveles b) Marco de 20 niveles c) Marco de 33 niveles

FIGURA 4. Arreglo de los disipadores en los marcos reforzados

Distorsiones de Entrepiso de los Marcos

En las Figuras 5, 6 y 7 se muestran las distorsiones de entrepiso para los marcos de 10, 20 y 33 niveles, correspondientes a los marcos sin reforzar, y reforzados con disipadores y con contravientos. Se puede observar que para los marcos reforzados las distorsiones son menores que las distorsiones límite que establece el reglamento vigente (0.012H, H=altura de entrepiso).

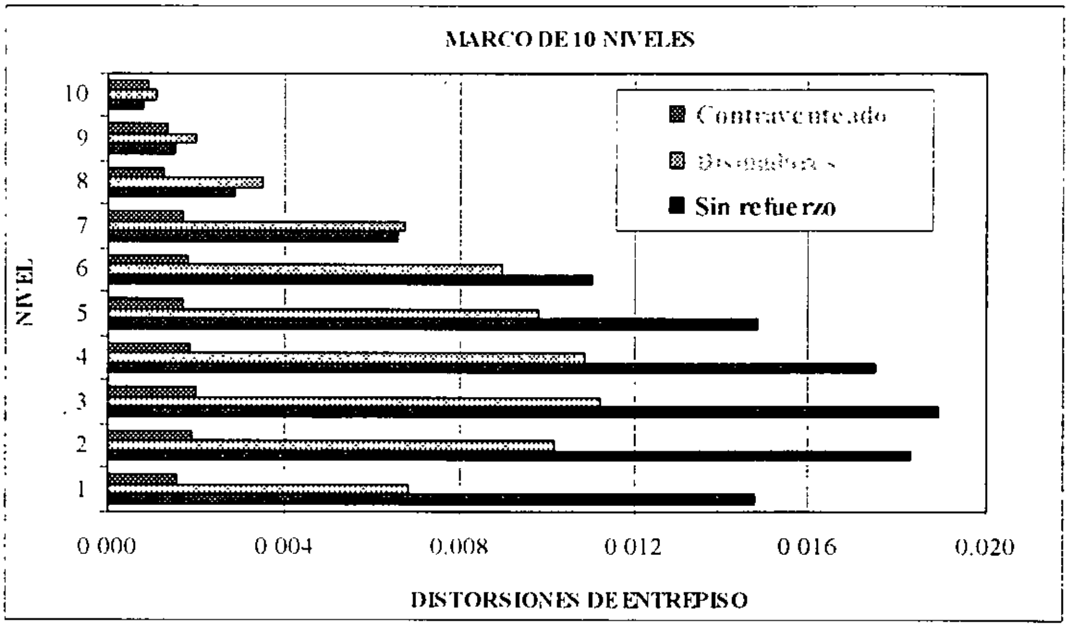


FIGURA 5 Distorsiones de entrepiso para los marcos de 10 niveles.

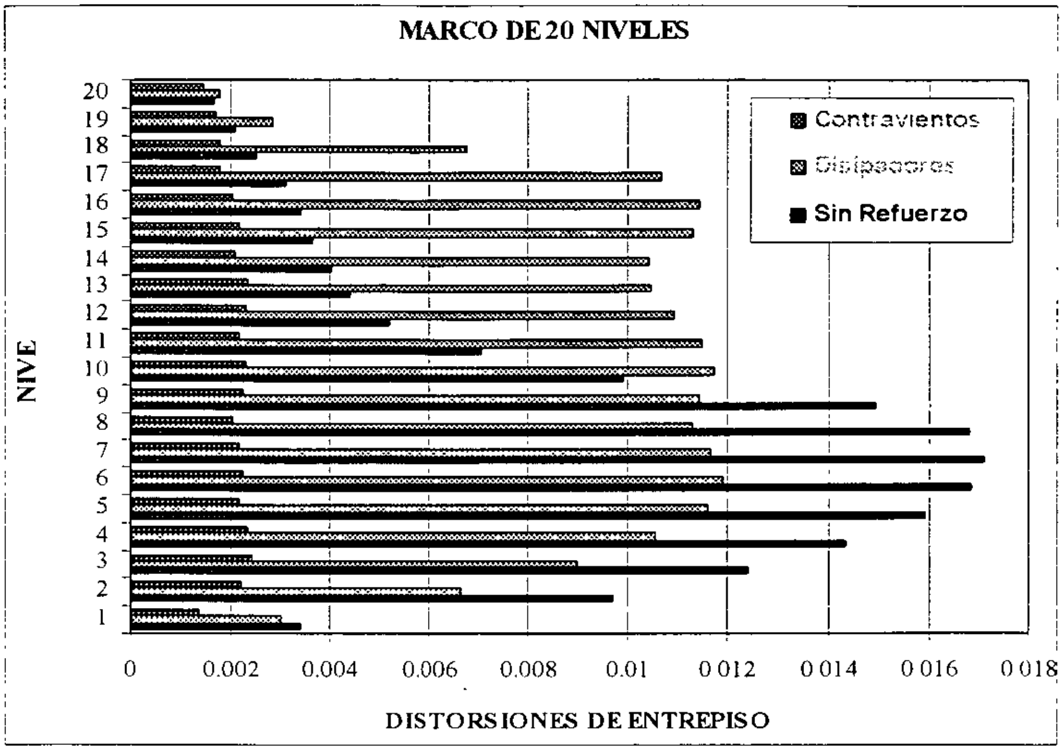


FIGURA 6. Distorsiones de entrepiso para los marcos de 20 niveles

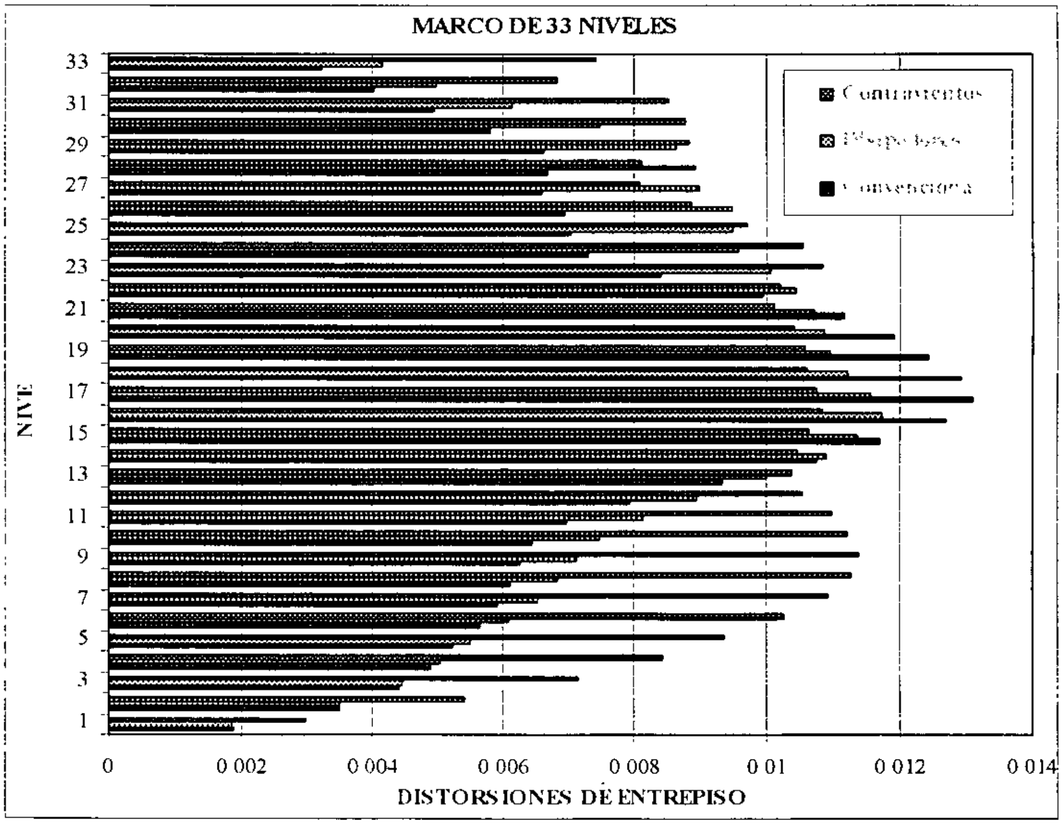


FIGURA 7. Distorsiones de entrepiso para los marcos de 33 niveles

PROCEDIMIENTO DE ANÁLISIS.

Respuestas de Interés.

Una vez que se diseñaron los respectivos refuerzos (usando contravientos o disipadores) se procedió al análisis dinámicos no lineal paso a paso de los marcos mediante el programa DRAIN 2D Las respuestas de interés obtenidas en el análisis son las siguientes:

1.- Coeficiente sísmico máximo (C_{sismo}) que demanda el sismo al edificio de múltiples grados de libertad (SVGDL). Este se obtiene a partir de la historia en el tiempo del cortante en la base dividido entre el peso de la estructura. Se calcula a partir de un análisis dinámico no lineal

2.- Desplazamiento máximo absoluto de la azotea ($\delta_{max\ de\ azotea}$).

3.- Desplazamiento máximo de entrepiso dividido entre la altura de entrepiso ($\gamma_{max\ relativo\ entrepiso}$).

Por otro lado, a partir de un análisis estático no lineal ("push over") se obtiene lo siguiente:

4.- Ductilidad global (μ_{global}) desarrollada del SVGDL. Se obtiene al dividir el desplazamiento máximo permitido (0.012H) entre el desplazamiento de fluencia (dy). Se calcula a partir de un análisis "push-over" usando el programa el Drain 2DX.

5 - Coeficiente sísmico que resiste la estructura (C_{res}) Este se obtiene de la curva que relaciona al coeficiente sísmico (fuerza cortante basal entre peso de la estructura) versus el desplazamiento de azotea, correspondiente a un desplazamiento igual a 0.012H, (H= altura del edificio)

Procedimiento General de Análisis

Enseguida se describen los pasos empleados en este estudio Estos se realizan para cada uno de los marcos

1.- Encontrar el C_{sismo}, C_{res}, $\delta_{max\ de\ azotea}$, $\gamma_{max\ relativo\ entrepiso}$, μ_{global} y T_o del SVGDL.

2.- Construir el espectro de respuesta de coeficientes sísmicos para ductilidad igual a uno, correspondiente al sismo usado como excitación (en este caso SCTEW85).

3.- Encontrar mediante iteraciones la demanda de ductilidad del sistema equivalente (μ^{equi}) Dicha demanda se obtiene mediante iteraciones. Debe satisfacerse el coeficiente sísmico basal (C_{sismo}) para un sistema de un grado de libertad con periodo T_o. Con esto conocemos el nivel de ductilidad desarrollado por el sistema equivalente de un grado de libertad (SE1GDL)

4.- Comparar el valor de μ^{equi} (SE1GDL) con μ_{global} (SVGDL)

5.- Comparar C_{sismo} asociada al SE1GDL con C_{res}, obtenido a partir del SVGDL.

Resultados Para el Marco de 33 Niveles

Siguiendo el procedimiento antes explicado, se construyó la Figura 8. Esta se asocia con el marco de 33 niveles sin reforzar (SR) con periodo inicial To = 3.07s, con el reforzado con disipadores (Dis) con To = 2.78s y con el reforzado con contravientos (Cv) con To = 1.83s. Esta figura muestra los espectros de coeficientes sísmicos correspondientes a las ductilidades equivalentes desarrolladas (μ^{equi}). En la figura también se indica (con flechas) el cambio que sufren los coeficientes sísmicos del marco cuando se refuerza con disipadores (Dis) o con contravientos (Cv).

En la Tabla 2 se muestran cada una de las variables definidas anteriormente. A partir de esta tabla podemos comparar los valores C_{sismo} con C_{res}, y μ^{equi} con μ_{global}.

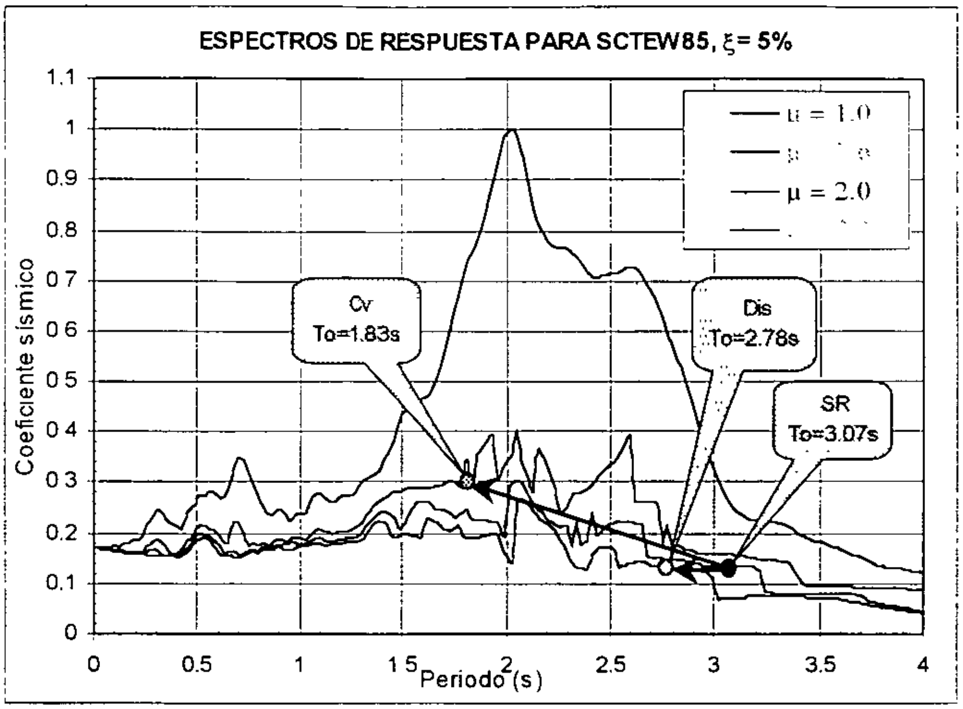


FIGURA 8. Representación gráfica de los coeficientes sísmicos demandados correspondientes a los edificios de 33 niveles

TABLA 2. Tabla comparativa de resultados del SE1GDL y del SVGDL para el marco de 33 niveles

MARCOS DE 33 NIVELES							
	RESPUESTA DEL SE1GDL			**RESPUESTA DEL SVGDL**			
MARCO	T_0	μ^{equi}	C_{sismo}	C_{res}	$\gamma_{max\ relativa\ entrepiso}$	$d_{max\ azotea}$ (cm)	μ_{global}
Sin Reforzar	3.07	2.0	0 13	0.134	0.0131	79.81	2 20
Con Disipadores	2 78	2.3	0.13	0.147	0.0117	88.90	2 30
Con Contravientos	1.83	1.63	0.30	0.329	0.0114	106 78	2 67

En la Tabla 2 se puede observar que el edificio convencional de 33 niveles con periodo fundamental T_0=3.07s y coeficiente sísmico desarrollado igual a C_{sismo}=0.13, presenta una demanda de ductilidad equivalente μ^{equi} =2.0, tal como se representa en la Figura 8. La demanda de ductilidad del SE1GDL μ^{equi} encontrada es del orden de la ductilidad global desarrollada por el SVGDL (μ_{global} =2.20). En este caso la relación entre estos valores es de 2 2/2.0 = 1.10. Para el caso con disipadores dicho factor es unitario y para el caso de contravientos se observa que la demanda de ductilidad equivalente (μ^{equi}) es muy diferente a la que se obtiene del análisis del SVGDL. Para este caso la relación entre demandas de ductilidad es de 1 64

La Tabla 2 también muestra que los coeficientes sísmicos resistentes (C_{res}) son mayores que los demandados por el sismo (C_{sismo}). Se deduce que sus resistencias son adecuadas. Así mismo, se muestran las distorsiones máximas relativas de entrepiso ($\gamma_{max\ relativa\ entrepiso}$). Nótese que para el marco convencional dichas distorsiones son mayores que las permitidas por el RCDF93 (0.012H), sin embargo, al reforzarse el edificio las distorsiones necesariamente son menores que 0.012H

Resultados de los tres marcos

a) Coeficientes sísmicos demandados (C_{sismo}).

La figura 9 muestra las respuestas de los marcos de 10, 20 y 33 niveles sin reforzar (SR), reforzados con disipadores (Dis) y reforzados con contravientos (Cv) en función de sus coeficientes sísmicos demandados (C_{sismo}), sus periodos originales (To) y finales. Estos corresponden con las ductilidades equivalentes desarrolladas (μ^{equi}) Como referencia se dibuja el espectro de coeficientes sísmicos para ductilidad igual a uno. Para ilustrarlo esquemáticamente se indica mediante flechas el incremento que tienen los coeficientes sísmicos entre ellos y a su vez el corrimiento de los periodos. Este incremento también se indica numéricamente en la Tabla 3.

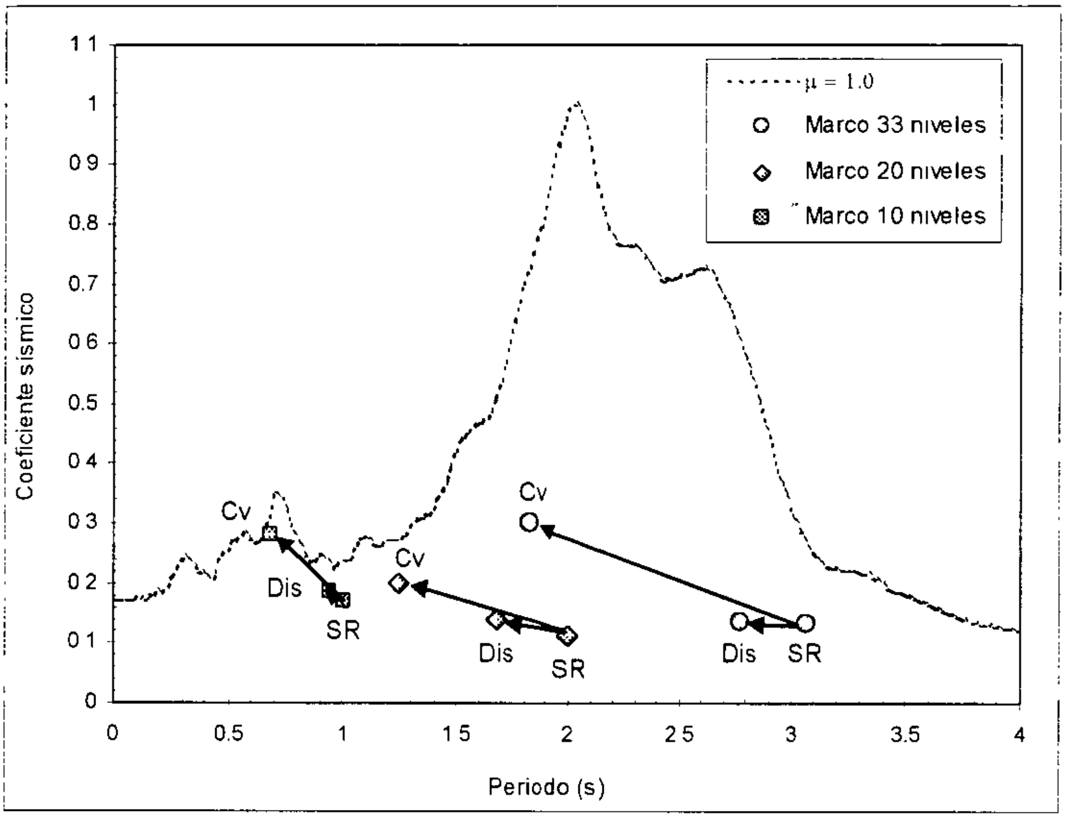


FIGURA 9. Representación gráfica de los coeficientes sísmicos demandados para los marcos sin reforzar (SR), reforzados con disipadores (Dis) y reforzados con contravientos (Cv).

TABLA 3 Relación de incremento de los coeficientes sísmicos

Relación entre los coeficientes sísmicos			
	Marco 10 niveles	Marco 20 niveles	Marco 33 niveles
Cv/SR	1.65	1.77	2.31
Dis/SR	1.09	1.24	1.00
Cv/Dis	**1.51**	1.43	**2.31**

En esta tabla se puede observar que la relación del coeficiente del marco de 10 niveles contraventeado entre el marco sin reforzar (Cv/SR) es de 1.65, es decir, que el sismo le demanda al marco con contravientos un incremento del cortante en la base de 1.65 veces el cortante que le demanda al marco sin refuerzo. Por otro lado, el incremento del cortante en la base en el marco con disipadores es 1.09 veces la del marco sin refuerzo. Por último, la relación del coeficiente correspondiente al marco con contravientos es 1 51 veces mayor con respecto al marco con disipadores.

La mayor relación Cv/Dis correspondiente a los tres casos de la Tabla 3 se presenta para el marco de 33 niveles reforzado con contravientos (Cv/Dis=2.31). Se deduce entonces, que el uso más eficiente de los disipadores en cuanto a reducción de la fuerza cortante basal se presenta para el marco de 33 niveles

Esto mismo se puede observar gráficamente en la Figura 9. En segundo lugar se encuentra el marco de 10 niveles, para el cual Cv/Dis = 1.51. Se hace notar que el marco de 10 y de 30 niveles se encuentran en zonas descendentes del espectro lineal de pseudoaceleraciones. Al reforzarlos estos presentan ordenadas espectrales mayores que los que tenían antes de ser reforzadas. Esto último sucede también con el marco de 20 niveles; sin embargo, el incremento en dichas ordenadas es menor

b) Distorsiones máximas de entrepiso ($\gamma_{max\ relativo\ entrepiso}$).

La figura 10 muestra las distorsiones de entrepiso correspondientes a los tres casos. El marco de 10 niveles sin refuerzo (SR) presenta distorsiones de entrepiso mucho mayores a las marcadas como límite (0 012H) por el RCDF93 (ilustrado con línea roja en la figura). Este marco reforzado con disipadores (Dis) presenta distorsiones por debajo de la línea que marca el límite tolerable. Estas distorsiones se obtienen intencionalmente cerca del límite con el propósito de que se tenga el mayor trabajo eficiente y la mayor absorción de energía del dispositivo disipador (al presentar mayor desplazamiento relativo de entrepiso) Mientras que las distorsiones de entrepiso para el marco reforzado con contravientos (Cv) se encuentran muy por debajo de la línea que marca el límite. En este caso el refuerzo con contravientos se diseña de tal manera que los contravientos resistan elásticamente la carga de compresión y tensión a las que se ven sometidos durante el sismo, evitando además el pandeo de los mismos. Los cambios de las distorsiones de entrepiso se ilustran mediante flechas en la Figura 10.

El comportamiento del marco de 20 niveles es muy similar al del marco de 10 niveles. Por otro lado, analizando el marco de 33 niveles sin refuerzo (SR) se observa que presenta distorsión poco arriba del límite de 0.012H y la del marco con disipadores está en el límite de la línea Hasta aquí el comportamiento es parecido a los dos casos anteriores (10 y 20 niveles); sin embargo, la mayor diferencia se encuentra en el marco reforzado con contravientos para el cual las distorsiones máximas de entrepiso son muy grandes comparadas con las de los marcos de 10 y de 20 niveles reforzados también con contravientos (Cv)

La relación de las distorsiones máximas de entrepiso para cada caso se presentan en la Tabla 4.

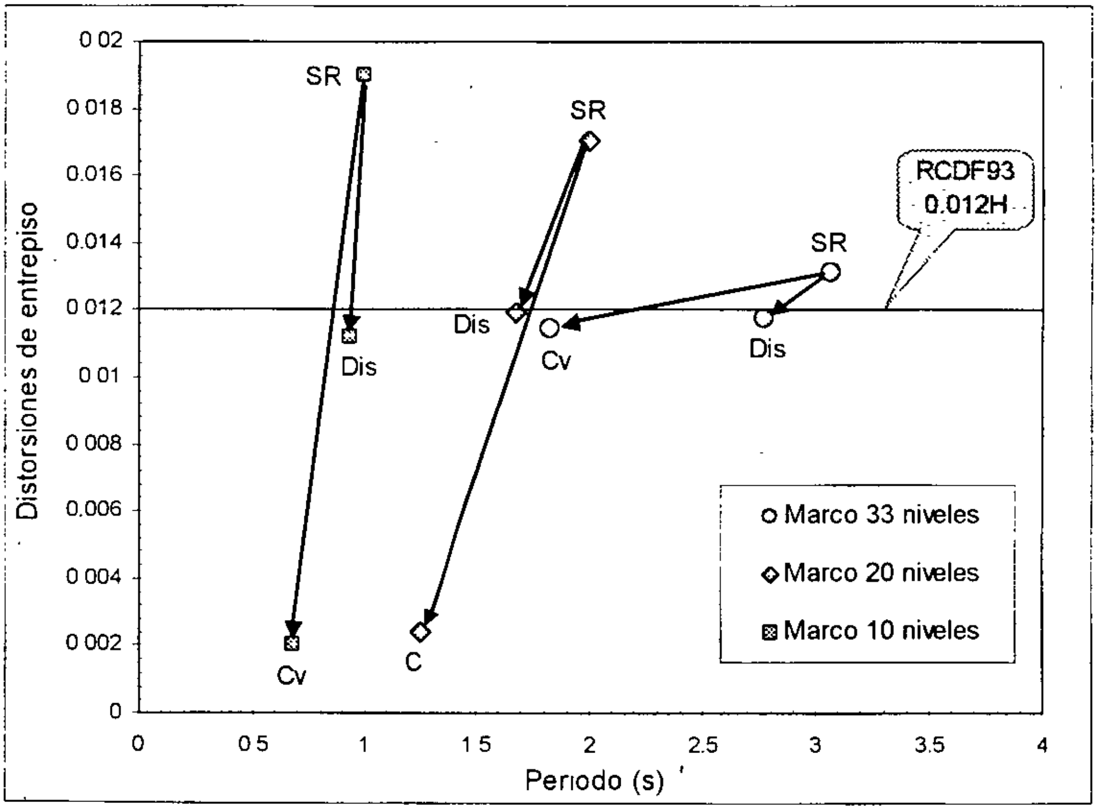


FIGURA 10. Representación gráfica de las distorsiones máximas de entrepiso para los marcos sin reforzar (SR), reforzados con disipadores (Dis), y reforzados con contravientos (Cv).

TABLA 4. Relación entre las distorsiones máximas de entrepiso.

	Marco 10 niveles	Marco 20 niveles	Marco 33 niveles
Cv/SR	0.11	0.14	0.87
Dis/SR	0.59	0.70	0.89
Cv/Dis	0.18	0.20	0.97

A partir de la Figura 10 y de esta tabla se puede observar que las distorsiones máximas de entrepiso son mayores para los marcos con disipadores que para los marcos contraventeados. Las menores relaciones de distorsiones se presentan para los casos de 10 y 20 niveles (Cv/SR = 0.11 y 0.14 respectivamente) Para el marco de 33 niveles esta relación es más grande (Cv/SR = 0.87). También se puede ver que la distorsión del marco de 33 niveles con contravientos es prácticamente igual a la del marco con disipadores (Cv/Dis = 0 97) No sucede así para los otros dos casos, en donde Cv/Dis = 0.18 y 0.20 correspondientes a los marcos de 10 y 20 niveles, respectivamente.

CONCLUSIONES

En la Tabla 3 se puede observar que el marco de 33 niveles con contravientos es el que presenta la mayor relación de coeficiente sísmico Cv/Dis = 2.31. Es decir, que reforzar con disipadores en este caso constituye una solución eficiente para reducir el cortante en la base. Esto es especialmente importante para el edificio con problemas de capacidad en su cimentación.

A partir de la Figura 10 en donde se presentan las distorsiones máximas de entrepiso de los tres marcos, se deduce que los contravientos añadidos al marco de 33 niveles suministran suficiente rigidez lateral a la estructura (el cambio del periodo fundamental de vibración es significativo); sin embargo, la reducción en distorsiones máximas es poco eficiente (menor que para los marcos de 10 y 20 niveles).

Para los casos aquí analizados se concluye que reforzar con disipadores es más eficiente en las estructuras de periodo largo, correspondientes a estructuras con periodos de vibrar ubicados en la zona descendente del espectro Por otro lado, reforzar estas estructuras con contravientos puede dar lugar a fuerzas basales grandes que podrían ser excesivas para edificios con problemas de capacidad en su cimentación

AGRADECIMIENTOS

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CURSOS ABIERTOS

XXVI CURSO INTERNACIONAL DE INGENIERIA SÍSMICA

MOUDLO V: DISEÑO SISMICO DE EDIFICIOS

TEMA

EDIFICIOS EN LOS QUE CONVIENE REFORZAR CON DISIPADORES EN ENERGIA EN LUGAR DE UTILIZAR CONTRAVIENTOS

EXPOSITOR: DRA. SONIA E. RUIZ GOMEZ
PALACIO DE MINERIA
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Performance-Based Design Approach for Seismic Rehabilitation of Buildings with Displacement-Dependent Dissipators

Sonia E. Ruiz and Hiram Badillo

A performance-based approach for seismic retrofitting of buildings with energy dissipating devices is presented The approach also may be seen as an *algorithm* useful for converging to a first approximation of a system to be analyzed according to the time history approach recommended in NEHRP Guidelines (FEMA 273, 1997). The algorithm is based on the analysis of equivalent SDOF models with added parallel elements that represent the dissipating systems The combined systems are analyzed under sets of accelerograms associated with different return intervals. The acceptance criteria are intended to control the interstory peak drift of the rehabilitated structure and the maximum ductility demand of the dissipating devices. The approach is successfully applied to a ten-story three-bay frame. The approach needs smaller computer time process as compared to that needed when multi-degree-of-freedom systems are used for the time history analysis. This saving in computer time makes attractive the algorithm proposed

INTRODUCTION

There is a consensus among structural design experts and code writers that in the near future most of the seismic design regulations will be established under a performance-based framework (Fafjar y Krawinkler, 1997) Also, there is a general agreement that the design tendency is to control the structural response deformations, instead of the resistance of the structures (Priestley, 1998). As a consequence, there is a need to develop new methods of analysis from the point of view of *performance-based* as well as *deformation control design*.

In this paper a *performance-based* approach for rehabilitation of buildings with passive energy dissipation devices is proposed. The approach presented in this paper also may be seen as an *algorithm* useful for converging to a first approximation of a system to be analyzed according to the time history approach recommended in NEHRP Guidelines (FEMA 273, 1997) Although the algorithm is applied to the rehabilitation of buildings with energy dissipators, it may also be adopted for retrofitting of buildings using conventional bracing members.

For the seismic rehabilitation of buildings, the NEHRP Guidelines (FEMA 273, 1997) recommend linear as well as nonlinear analysis procedures One of the nonlinear dynamic procedures is the time-history approach This involves the step-by-step analysis of the

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response time history of a multi-degree-of-freedom model of the building. This commonly represents a long computer time. On the contrary, the use of equivalent SDOF systems, as proposed in this paper, reduces the computer time of analysis considerably, as compared to that needed for MDOF systems.

The algorithm proposed is illustrated by means of a ten-story three-bay reinforced concrete frame rehabilitated with hysteretic steel dissipators. For these, the force-displacement response is a function of the relative displacement between each end of the dissipation system

In order to control the deformation of the structure studied here, some assumptions had to be made about the tolerable maximum drifts of the building and about the allowable maximum ductility demands on the dissipators, for the intensities corresponding to different return intervals. The need to define the performance levels associated with different seismic hazard levels has been pointed out by experts in structural design (Krawinkler and Fafjar, 1997). At this time there are many efforts oriented in this direction in different regions of the world.

DESIGN APPROACH

The approach proposed uses single-degree-of-freedom (SDOF) equivalent systems for different assumptions about the characteristics of the dissipators The time histories of the responses of these systems under the action of scaled motions are studied by means of step-by-step analysis The acceptance conditions required in this step is intended to *control the deformations* of the structure. The approach may be applicable to different performance levels (from "fully operational" to "collapse prevention" levels).

In what follows the steps of the algorithm proposed are listed In the next section, each of these steps is extended, and the applicability and limitations of the approach are discussed.

a) An equivalent SDOF model of the system to be rehabilitated is determined first This model is characterized by its mass M_c*, lateral initial stiffness K_c* and lateral strength R_c*. Some methods have been proposed in the literature to find the equivalent SDOF system (Miranda, 1991; Qi and Mohele, 1992; Collins, Wen and Foutch, 1995). The degradation of the stiffness and strength for cyclic load excitation can be taken into account as discussed later.

b) An energy-dissipating element with lateral stiffness K_d and strength R_d is added (in parallel) to the equivalent SDOF system The ratios of the corresponding mechanical properties of the dissipating system and the conventional structure are represented by the non-dimensional parameters $a_o = K_d/K_c$ and $b_o = R_d/R_c$.

c) The combined SDOF system is excited with a family of earthquakes with statistical properties similar to those of the design earthquake. All motions in the set correspond to the same return interval, T_R

d) The dynamic response is determined by means of a time-history analysis. The responses of interest are the peak values of the interstory drift (relative lateral displacement divided by the story height = $(\delta/h)_{max}$), and the maximum ductility demands on the dissipator devices $(\mu_d)_{max}$.

e) Adequate probability distribution functions are fitted to the variables $(\delta/h)_{max}$ and $(\mu_d)_{max}$. Then the nominal values $(\delta/h)^*_{max}$ and $(\mu_d)^*_{max}$ that correspond to a given probability of exceedance p* are calculated. The selection of p* is discussed later.

f) Steps b, c, d and e are repeated, assuming different values of the stiffness and of the strength of the dissipator system, this means, different values of a_o and b_o. It is noticed that, for the type of dissipators treated here, once that a_o is selected, the value of b_o is given. The ratio between a_o and b_o, as well as the assumptions behind, are explained later. The results at this stage of the process can be represented as in Fig. 1 In this, the vertical left axis represents the peak interstory drift and the right axis is the maximum ductility demand developed by the dissipator

g) Steps b, c, d, e and f are repeated for different motions associated with different return intervals T_R A graph similar to Fig 1 can be drawn for the intensity that corresponds to each return interval. It is noticed that the properties of the equivalent SDOF system depend on the assumed drift ratio.

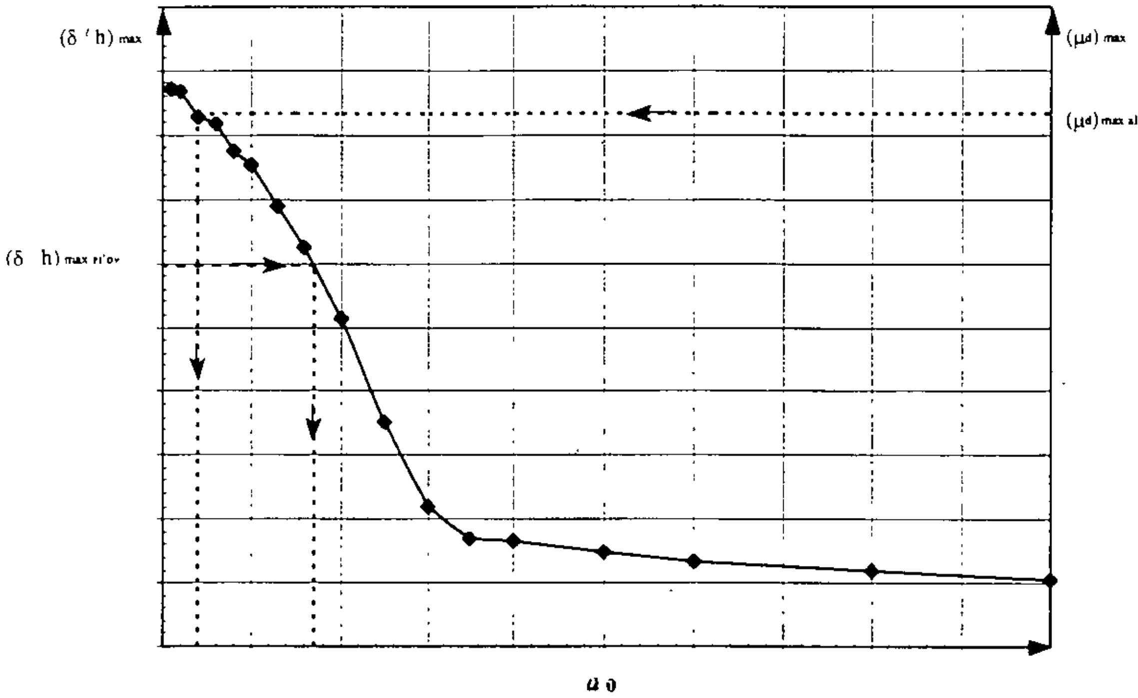


Figure 1. Nominal values of maximum drifts $(\delta/h)_{max}$ and ductility demands $(\mu_d)_{max}$ as functions of a_0 The curve corresponds to a probability of exceedance equal to p* for a ground motion intensity associated with a specified return interval.

h) The values $(\delta/h)_{max}$ and $(\mu_d)_{max}$ associated with different values of a_o and with different return intervals T_R may also be represented by bi- or tri-dimensional plots similar to those shown in Figs. 2 and 3.

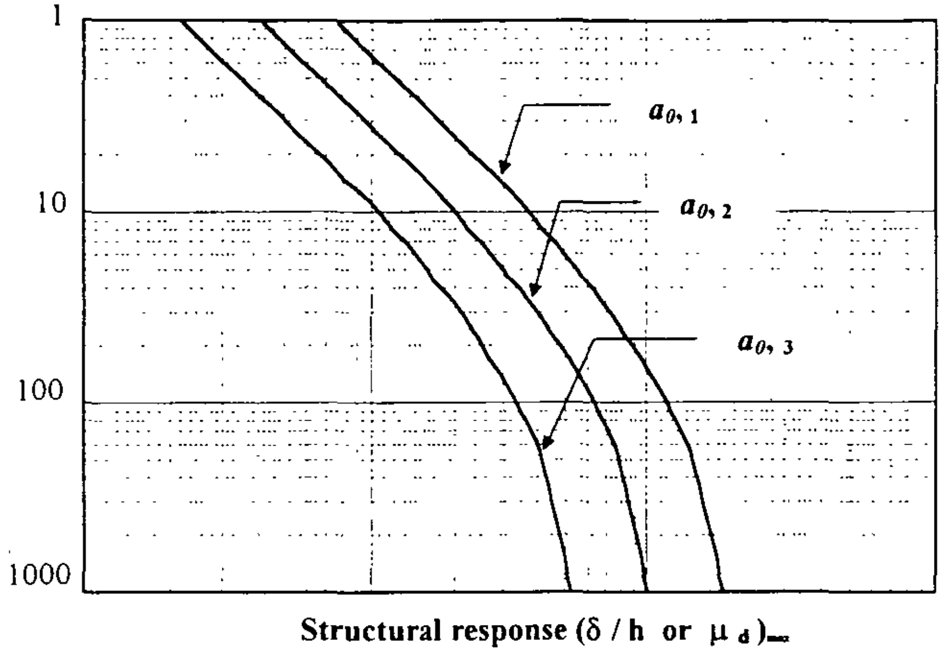


Figure 2. Response of the SDOF systems with dissipators associated with a specified probability of exceedance, for different values of a_0 and different return intervals.

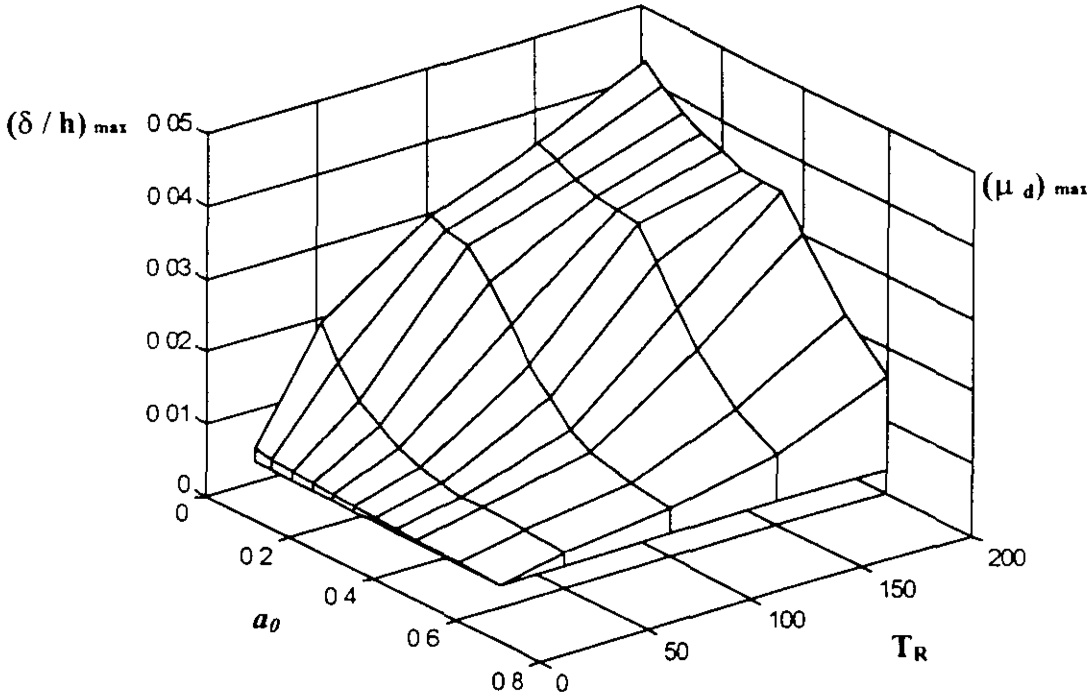


Figure 3 Response of the SDOF systems with dissipators associated with a specified probability of exceedance, for different values of a_0 and different return intervals.

i) From these figures, a value of a_o is selected for each return interval, in such a manner that it corresponds to the allowable peak interstory drift of the structure and to the maximum ductility demand of the dissipators for that return interval. The allowable values corresponding to the different T_R are supposed to be taken from a performance-based code. The largest value selected of a_o determined in this manner governs the design.

j) The value of a_o obtained in this way defines the initial trial for the design of the rehabilitated MDOF structure At this stage of the analysis it is necessary to decide about the distribution of the dissipators along the height of the structure This is discussed later.

k) Verify for the MDOF system that the peak interstory drift of the building and the maximum ductility demand of the dissipator system are within tolerable values when the rehabilitated structure is subjected to three critical ground motion time histories with a specified intensity. The selection of these is discussed later.

l) Verify that the accumulated plastic hinge rotations on the structural members are smaller than or equal to their rotation capacities.

m) If necessary, make some adjustments on the number or on the distribution of dissipators along the height of the structure and repeat steps k and l.

COMMENTS ABOUT THE DESIGN APPROACH

This section makes some reflections and considerations about each of the steps mentioned above.

REPRESENTATION OF THE EQUIVALENT SDOF MODEL (STEPS a AND b MENTIONED IN THE PREVIOUS SECTION)

As it is well known, the seismic response of a structure is affected, among many other factors, by the degradation in the mechanical properties of its structural elements (Gupta and Krawinkler, 1998) Due to this, it is important to consider these effects in the response of SDOF equivalent systems, at least by means of simplified models. On the other hand, the cyclic behavior of the dissipation elements based on the deformation of metals is very stable and does not show any strength or stiffness degradation

An equivalent system that accounts for stiffness degrading can be developed applying to the original structure a low-frequency sinusoidal excitation with increasing amplitudes, similarly to a typical pseudo-dynamic laboratory test with controlled displacements. The constitutive function for the equivalent system can then be derived on the basis of the calculated cyclic response curve expressed in terms of base-shear versus roof-displacement of the detailed system.

The mathematical model can be represented as shown in Fig. 4 for a ten-story three-bay frame. All mass is removed from the structure, and a rigid element (hinged at its base) with large mass is added. The vertical element is attached to each floor level by very rigid springs. A variation of this model, but using monotonically applied static loads and with springs arranged according to the desired static lateral load distribution, was proposed by Moehle and Alarcón (1986)

SEISMIC HAZARD AND SCALED GROUND MOTIONS (STEP c)

In the performance-based formulation the selection of the return interval T_R of seismic events with intensities equal to or greater than a certain level will depend on the objectives of the rehabilitation. These objectives are selected thinking on the total costs and on the benefit to be obtained The goal of the rehabilitation project may be to satisfy one or more objectives This may imply using for the analysis N design motions, each associated with a given return interval $(T_R)_i$, i =1,N.Body Text

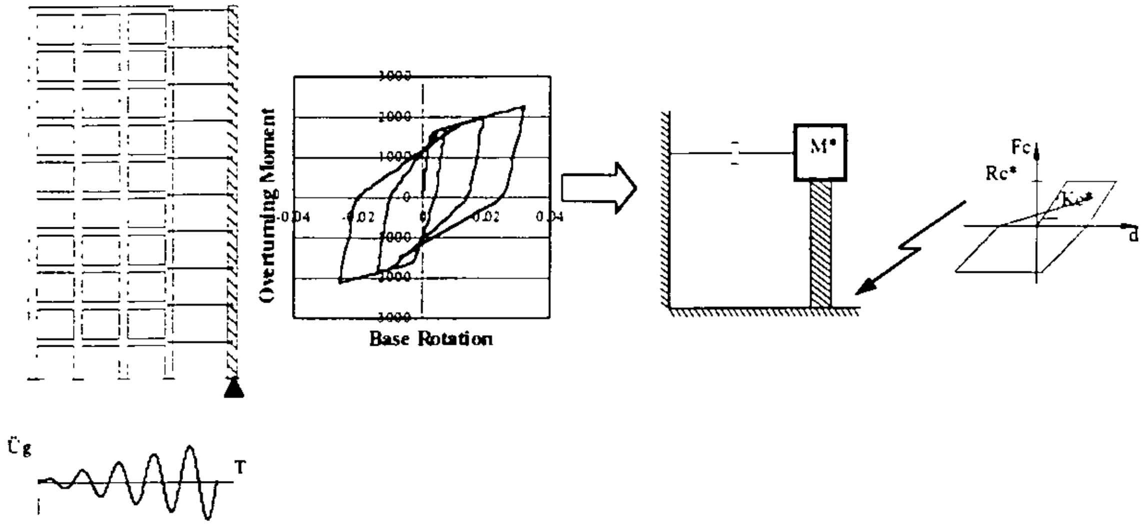


Figure 4. Representation of the approach to obtain the SDOF equivalent system considering degrading stiffness of the structural elements under cyclic loads

The frequency content of non-characteristic earthquakes (earthquake motions corresponding to a small T_R) may be different from that associated with high-intensity motions (corresponding to characteristic earthquakes). A realistic analysis must take into account these differences.

In the example presented in this paper the SDOF systems are analyzed under motions corresponding to six different return intervals; however, as it will be seen later, only three of them are taken as references for establishing acceptance criteria

For each return interval a set of ground motions is used as base excitation. The motions (real or simulated) can be scaled in such a way that their spectral ordinates for the fundamental period of the system analyzed are equal to that associated with the specified return interval. Larger standard deviations of the response may be obtained if other scaling criteria are used.

DYNAMIC RESPONSES OF THE EQUIVALENT SDOF SYSTEMS WITH DISSIPATORS (STEP d)

Seismic damage on a building is usually controlled by limiting the interstory drifts or the ductility demands that result from the response to the design earthquake There are other structural-response variables, such as the maximum floor velocity or floor acceleration, that are more directly related to damage on some non-structural elements or facility contents However, the study presented here deals only with buildings for which structural damage is determined by the maximum intersotry drift $(\delta/h)_{max}$

The aim of the approach proposed is to limit damage by controlling $(\delta/h)_{max}$ and $(\mu)_{max}$ for different ground motion intensities

PROBABILITY OF EXCEEDANCE OF THE CONTROL VARIABLES (STEP e)

The selection of the probability of exceedance p* of the control variables $((\delta\ /h)_{max}$ and $(\mu)_{max})$ depends, among other factors, on the consequences of the structural failure. As this probability becomes larger the reliability of the design increases. The approach presented here assumes that this probability value is kept constant for the whole range of a_o values assumed in the analysis.

In the example of this paper, peak deformation and maximum ductility demands are considered as random variables with lognormal probabilty distribution. The probability of exceedance of the design value is p* = 2 per cent.

RELATION BETWEEN THE STIFFNESS AND THE STRENGTH RATIOS (STEP f)

The a_o and b_o values are related as follows.

$$a_o = b_o\,(\delta_{yc} / \delta_{yd}) \tag{1}$$

where δ_{yc} and δ_{yd} are the yield deflections of the elements that represent the conventional frame and the energy dissipating devices, respectively The ratio R_d/K_d is assumed to remain constant during the design process, because its value is determined by the dimensions of the components of the energy dissipating system and by the mechanical properties of the materials with which they are built

PERFORMANCE SURFACE AND TARGET PERFORMANCE LEVELS (STEPS g, h i)

The structural "performance surface" as that of Fig. 3 represents the values of structural response associated with a given probability of being exceeded during an earthquake with a specified intensity The ordinates defining this surface must be smaller than or equal to the structural capacities specified by the performance-based code. This must be satisfied over the entire range of values of structural response.

INFORMATION OBTAINED FROM THE SDOF MODEL (STEP j)

The results obtained from the analysis of SDOF models provide information about approximate average values of the stiffness and strength ratios $a_o = K_d/K_c$ and $b_o = R_d/R_c$ that must be adopted for the MDOF structure rehabilitation design Local variations of the mechanical properties of the dissipators along the building height must be determined next.

In general it will be convenient to assume that the dissipators will be placed on those stories with values of the maximum interstory drift larger than the allowable value However, it should be verified that no concentrations of rotation ductility demands occur on structural members in the vicinity of the retrofitted stories were the dissipators are placed If this were the case, the dissipators should be installed starting from the base of the frames up to the stories where the dissipators are required to reduce lateral drift. Normally the dissipator devices are needed at the lower and at the middle portion of a building, rather than at the top. As an initial trial it may be assumed that the dissipator stiffness value is $K_d = a_o \underline{K}_c$, where $\underline{K}_c$ represent the mean value of the lateral stiffness of the stories where the dissipators will be placed

VERIFYING THE ACCEPTANCE CRITERIA (STEP k)

The MDOF structure with dissipators must be subjected to three critical motions in order to verify that $(\delta/h)_{max}$ and $(\mu)_{max}$ are smaller than the values specified by the code The return interval associated with these motions will be that corresponding to the governing situation. It is also necessary to verify that the accumulated hinge-rotation demands at the ends of beams and columns of the rehabilitated building are smaller than the corresponding capacities.

One way to select the three critical motions is by requiring that the responses $(\delta/h)_{max}$ and $(\mu)_{max}$ of the corresponding SDOF equivalent systems under these motions have probabilities of exceedance close to p*. (The selection of these motios will become clear through the example explained in next section, particularly when analizing Fig. 8.)

ILLUSTRATIVE EXAMPLE

Suppose that it has been decided to rehabilitate with hysteretic energy dissipators the structure shown in Fig. 5. The rehabilitation is needed because the drift demands at some stories are larger than the allowable value prescribed by the code. The main properties of the frame are summarized in Table 1. Its fundamental period of vibration is equal to 1s.

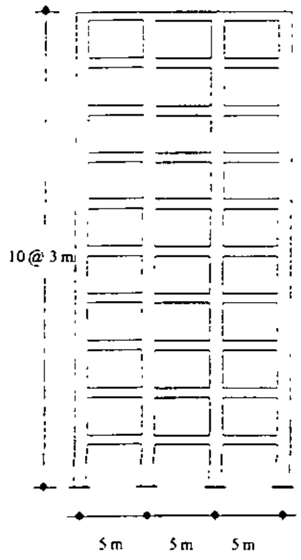


Figure 5. Geometry of the MDOF building to be retrofitted.

Table 1. Properties of the cross sections

Story	Column Section (m)	Girder Section (m)
1 to 4	0.56 × 0.56	0.35 × 0.75
5 to 6	0.54 × 0.54	
7 to 8	0 50 × 0.50	
9 to10	0.42 × 0 42	

8

The structure is supposed to be located at a soft soil site in Mexico City, where the seismic hazard may be represented by Fig 6.

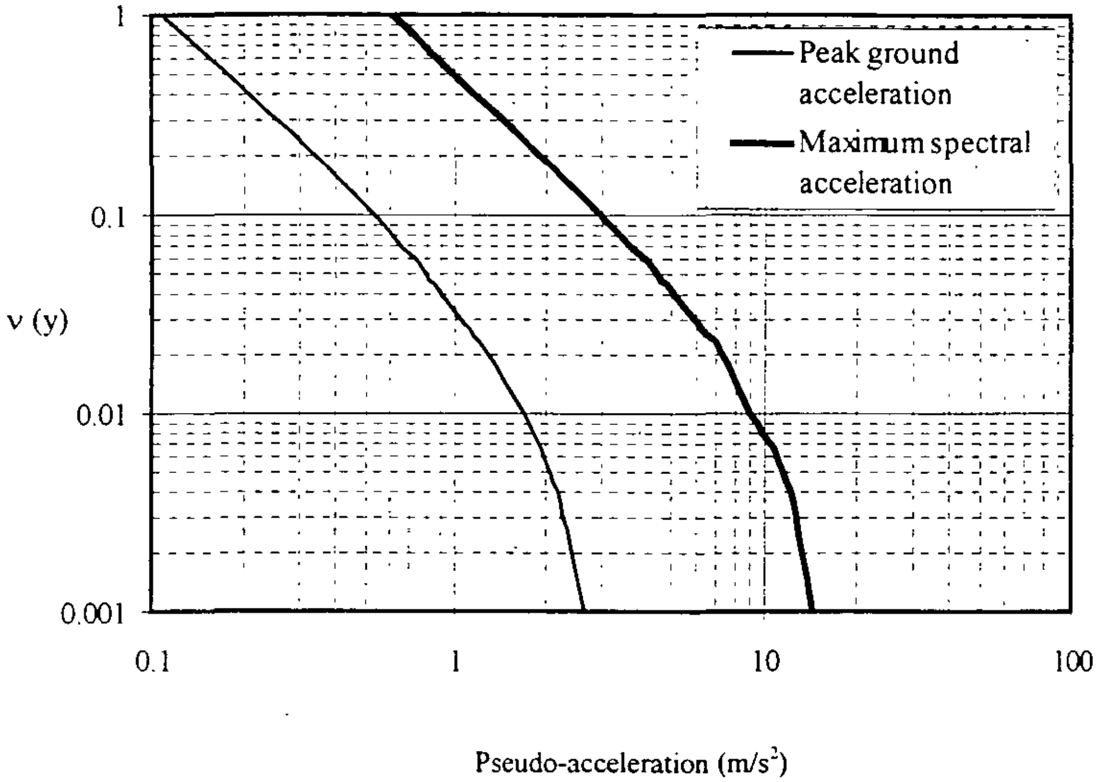


Figure 6. Intensity-recurrence curves for soft soil in Mexico City.

The properties of the SDOF equivalent systems for the frame to be retrofitted were obtained in accordance with the approach proposed by Collins et al (1995). These are as follows (assuming a drift ratio of 1%): K_c* = 1009.6 T/m, R_c* = 52 5 T, M_c* = 28 9 T-s^2/m, P* = 1.25 (scale factor for base motion) The degrading parameters obtained with the procedure explained previously (according with Fig. 4) are: α = 0.07 and β = 0 4 (Badillo, 2000). Here, α and β are the parameters of the extended version of Takeda's constitutive function (Kanaan and Powell, 1973) and the other variables were defined previously.

A calibration of the equivalence between MDOF and SDOF systems having non-linear behavior (assuming a drift ratio of 1%) was performed. A total of 30 simulated earthquakes were used for each case as base excitation. Both for maximum roof displacements and peak interstory drifts a good agreement is observed in Fig. 7a between the results obtained by means of the MDOF model and the SDOF system. Similar results were obtained for buildings containing energy dissipators (see Figs. 7b, Badillo, 2000).

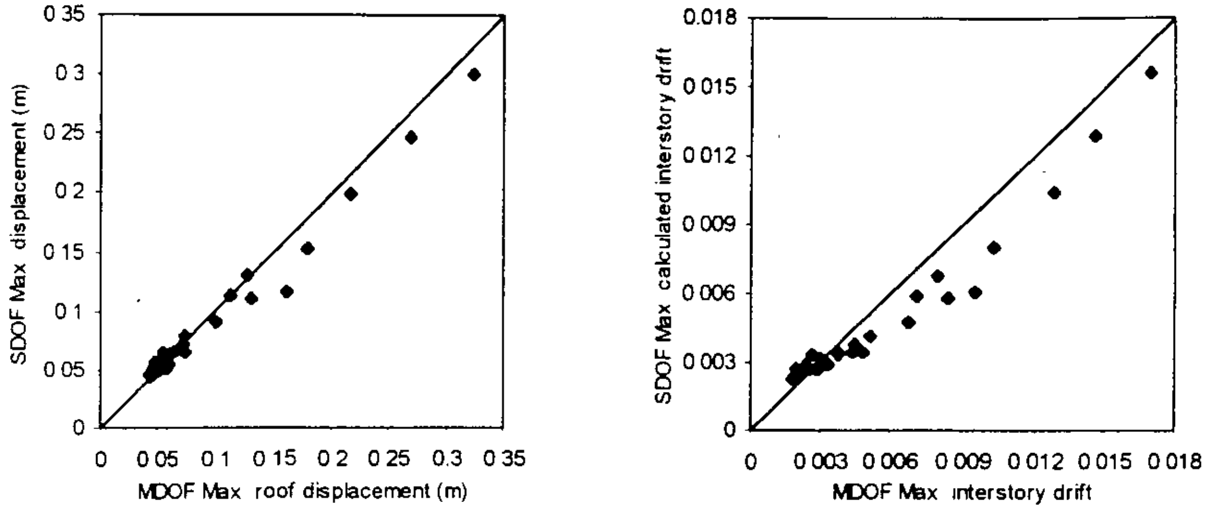


Figure 7.a. Scatterplot comparing maximum responses of roof displacement and interstory drift predicted by equivalent SDOF model with responses obtained from a MDOF model of the original frame

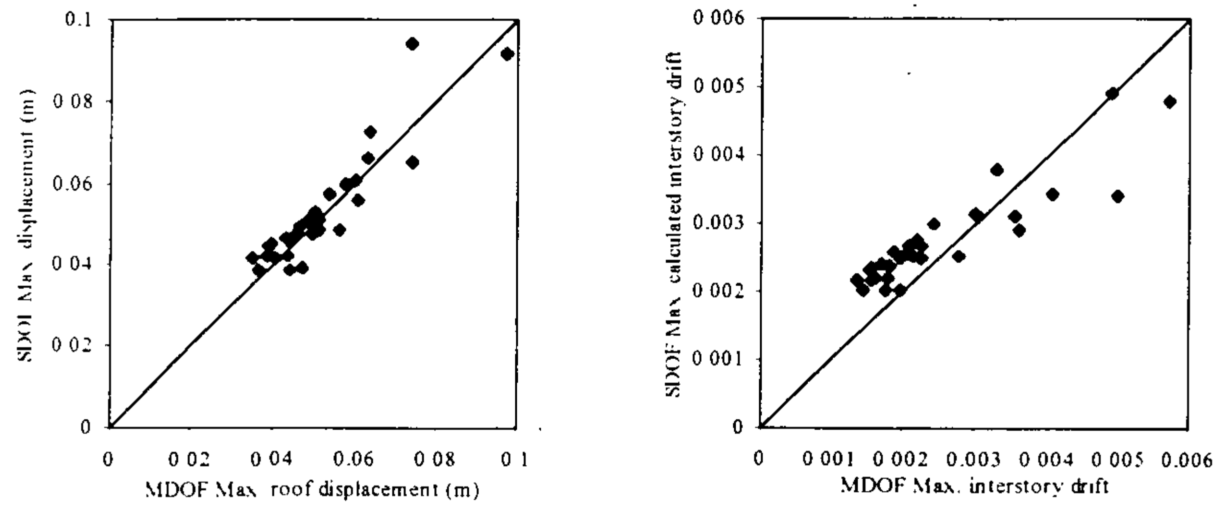


Figure 7.b. Scatterplot comparing maximum responses of roof displacement and interstory drift predicted by equivalent SDOF model with responses obtained from a MDOF model of the retrofitted frame with dissipators.

In this paper the dissipating devices are assumed to be displacement-dependent with bi or tri-linear hysteretic behavior. In this example, a bilinear element equivalent to the energy dissipator was added to the SDOF system mentioned above. The a_o parameter was varied from 0 001 to 0 6 Six return intervals were selected for the analysis: T_R = 20, 50, 100, 150, 200 and 250 years For each of these, 14 scaled simulated accelerograms were used as base excitation.

In this particular case the ground motions were scaled in such a manner that the maximum spectral acceleration was equal to that associated with the specified return interval A more appropriate scaling criterion would make the spectral ordinate for the *fundamental period* of the structure of interest equal to that associated with the specified return interval. Although the authors are aware of this, they could not scale the motions in this manner because they only had access to seismic hazard curves corresponding to peak ground acceleration and to maximum spectral acceleration (as shown in Fig. 6). However, this assumption does not affect the concept of the design approach.

The consequences of scaling the motions using the maximum spectral acceleration as the scaling parameter instead using the spectral ordinates for the fundamental period is that the standard deviation of the response becomes larger.

One of the graphs obtained (similar to Fig. 1) is shown for illustration in Fig 8. This corresponds to a return period $T_R = 50$ years. The left vertical axis represents the maximum interstory drift, and the right axis the maximum ductility demand on the dissipators With a continuous line are represented the responses associated with a probability of exceedance of 2% Lognormal probability density functions were fitted to these data. The large dispersion shown by the curves plotted in Fig. 8 is related to the scaling criterion used in the example.

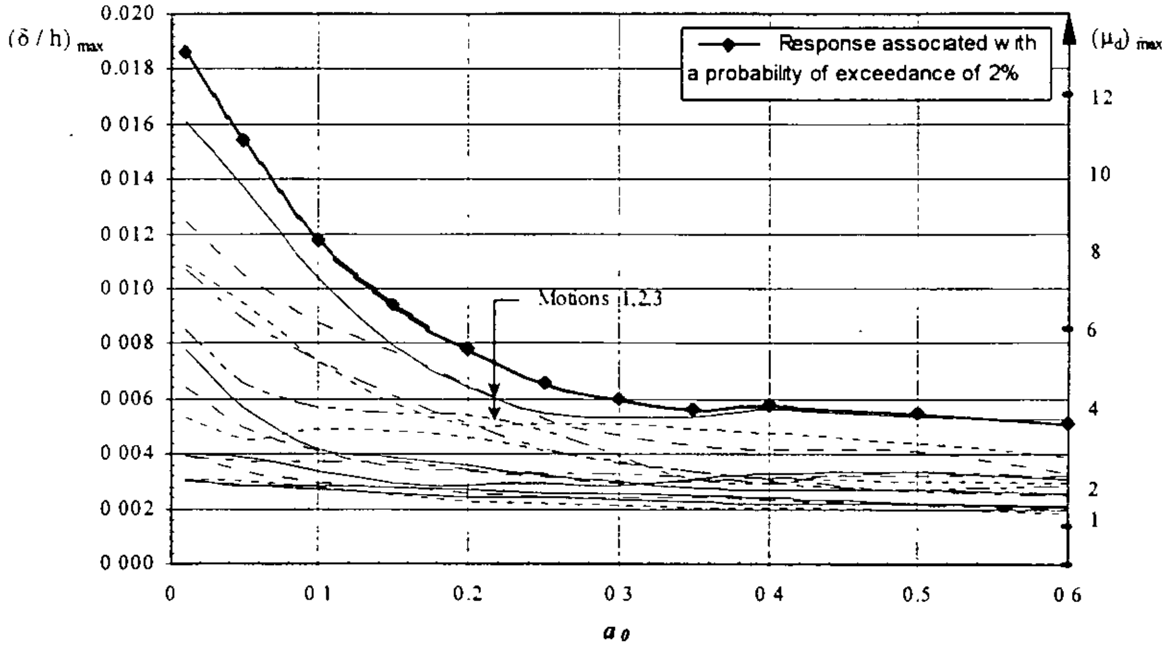


Figure 8. Response of the SDOF for different values of a_0 and its corresponding response associated with a probability of exceedance p* = 2%. $T_R = 50$ years.

An alternative way to see the results is through a three-dimensional representation as shown in Fig. 9. This representation has the advantage that the whole information can be analyzed from only one figure Again, in this case the "performance surface" is associated with a probability of exceedance p* = 2% Obviously, a different "performance surface" would be obtained if a different p* value were assumed.

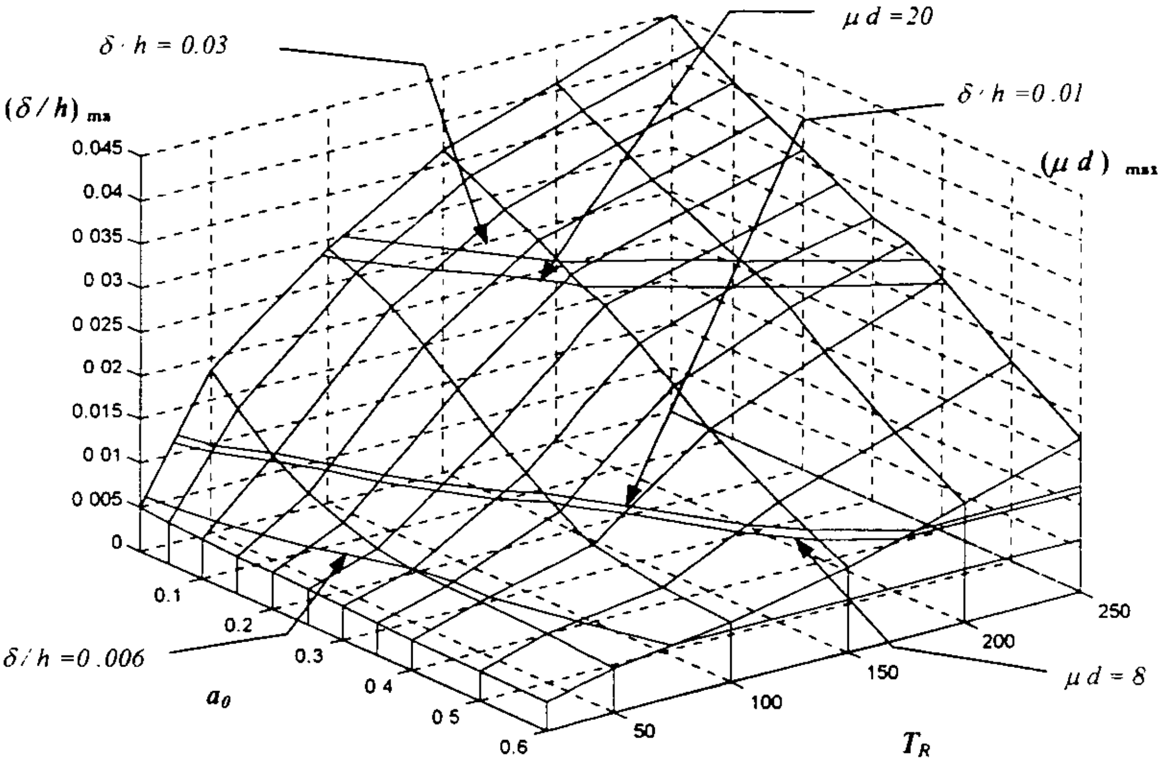


Figure 9. Performance surface

Notice that the "performance surface" can be related to initial costs of the rehabilitation solution. This surface can be compared with three-dimensional graphs similar to that of Fig. C2-2 of FEMA 274. As the structural performance grows, and the earthquake severity increases, the initial costs increase as well Similarly, from Fig. 9 it can be seen that as the a_o value becomes smaller (this means a smaller number of dissipators) and the intensity of the motion increases, the structural response becomes higher

One easy way of selecting the critical a_o value is through bi-dimensional graphs like those shown in Fig. 10 and 11. In these, the vertical axis represents the return interval T_R, and the horizontal is the response of interest $[\delta/h)_{max}$ and $(\mu)_{max}$] corresponding to a probability of exceedance of 2 per cent Each curve corresponds to an a_o value The limiting values specified by the code will determine the value of a_o that satisfies the design conditions.

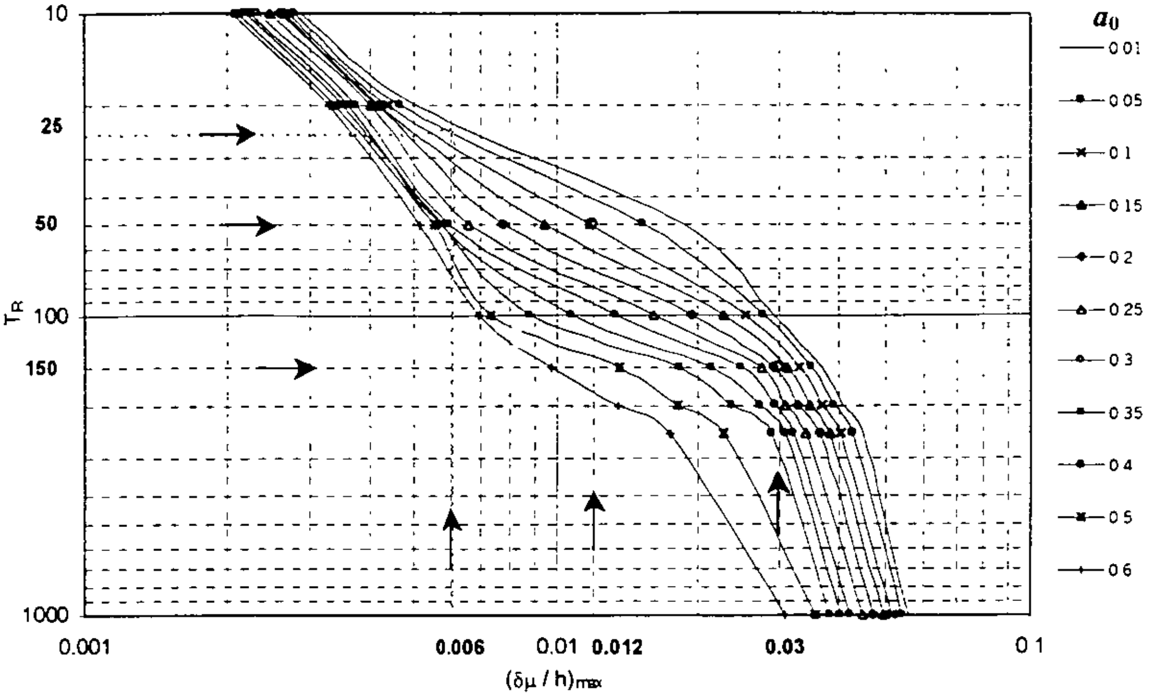


Figure 10. Story drift curves for MDOF systems for a probability of exceedance of 0.02, for motions with different return intervals T_R

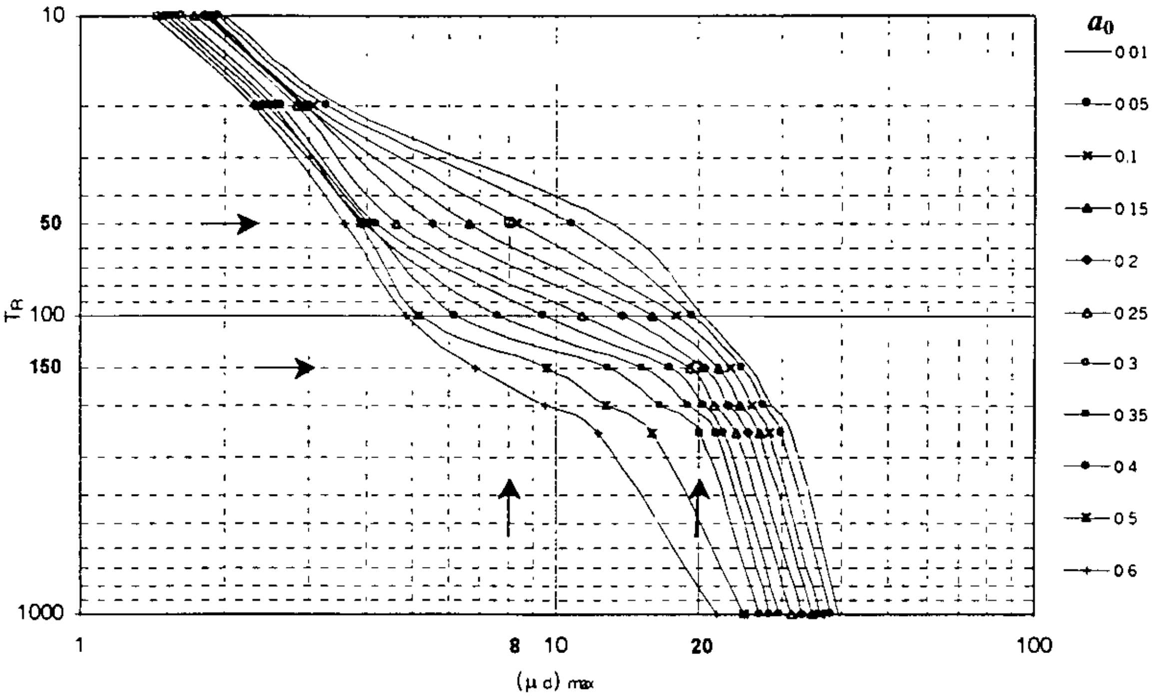


Figure 11. Ductility demand curves for the dissipators for a probability of exceedance p* = 0 02, for motions with different return intervals T_R

Limiting design values as shown in Table 2 were adopted in this example These values are used here only for illustration purposes and should not be taken as recommendations for design.

Table 2. Limiting design values used in the example

Return Interval, T_R (years)	Allowable $(\delta / h)_{max}$	Allowable $(\mu_d)_{max}$
25	0 006	-
50	0.012	8
150	0.030	20

Five values of a_o are selected by using the information on Table 2 and on curves in Figs. 10 and 11. The values chosen appear in Table 3. There it is observed that the condition that governs the design is $a_o = 0.22$, which is the highest value of a_o. This corresponds to the condition of maximum ductility demand of the dissipators, when the structure is subjected to a motion with return interval $T_R = 150$ years. .

Table 3 Values of a_0 for different return intervals T_R

a_0					
$T_R = 20$ years		$T_R = 50$ years		$T_R = 150$ years	
$(\delta / h)_{max} = .006$	0.009	$(\delta / h)_{max} = .012$	0 1	$(\delta / h)_{max} = .03$	0.18
		$(\mu_d)_{max} = 8$	0 11	$(\mu_d)_{max} = 20$	0 22

Under these conditions, the lateral stiffness of the dissipator system is estimated as $K_d = 0.22 \underline{K}_c$ In this case $\underline{K}_c = 24081$ T/m This is the mean value of the lateral stiffness corresponding to the lower six stories Six U type dissipators (Aguirre and Sanchez, 1992) were installed at each of the diagonal elements in the lower six stories. The distribution of the diagonal elements is shown in Fig. 12.

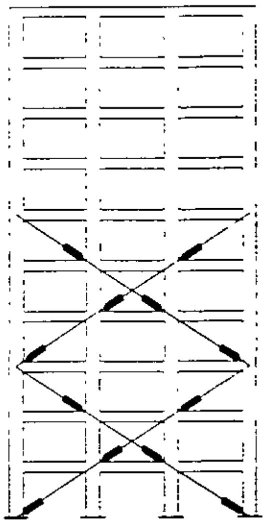


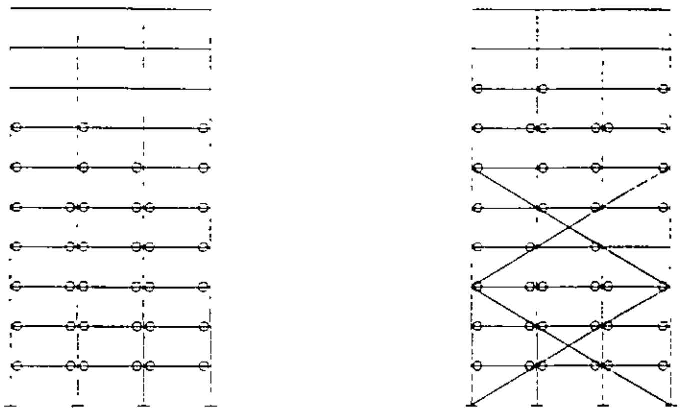
Figure 12. Arrange of the energy dissipators devices

The rehabilitated structure (Fig 12) was excited with the three scaled accelerograms named Motions 1, 2 and 3 in Fig. 8. These correspond to responses with exceedance probabilities p* close to 2 percent and a_o = 0.22. The maximum responses of the structure under Motions 1, 2 and 3, corresponding to return intervals of T_R = 25, 50 and 150 years are shown in Table 4 The results show that the maximum dissipator ductility demand for T_R = 150 years is close to the limiting value $[(\mu_d)_{max} = 20]$ given in Table 2.

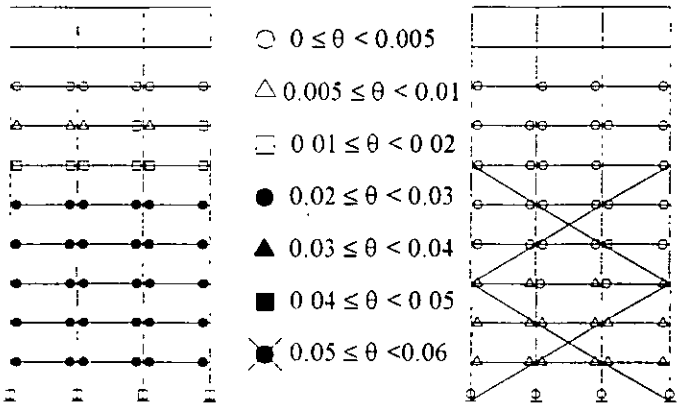
As a final step, the maximum cumulative plastic hinge rotations (θ_{max}) developed on the rehabilitated structure were obtained for the critical motion associated with return intervals of T_R = 25, 50 and 150 years They are shown in Fig. 13a-c. It was verified that the demanded rotations were smaller than the rotation capacities of the members.

Table 4. Response of the retrofitted frame for the limiting design values

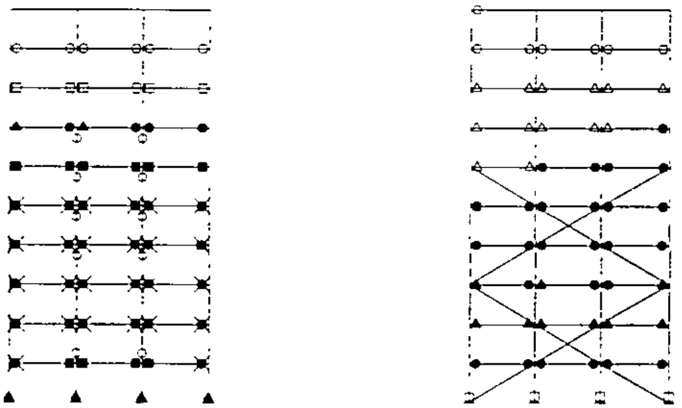
Interstory	T_R = 25 years		T_R = 50 years		T_R = 150 years	
	$(\delta / h)_{max}$	$(\mu_d)_{max}$	$(\delta / h)_{max}$	$(\mu_d)_{max}$	$(\delta / h)_{max}$	$(\mu_d)_{max}$
10	0.0010	-----	0.0011	-----	0.0021	-----
9	0.0018	-----	0.0018	-----	0.0069	-----
8	0.0021	-----	0 0022	-----	0 0141	-----
7	0 0025	-----	0 0029	-----	0 0175	-----
6	0.0020	1 61	0.0026	2.10	0.0183	15.00
5	0 0020	1.64	0.0035	2.89	0.0198	16.20
4	0 0024	1 96	0 0052	4.23	0.0218	17.84
3	0.0034	2 81	0 0072	5.86	**0.0236**	**19.31**
2	**0.0039**	**3.22**	**0.0076**	**6.25**	0.0230	18.85
1	0.0026	2.15	0.0048	3.95	0.0196	16.01



a) $T_R = 25$ years



b) $T_R = 50$ years



c) $T_R = 150$ years

Figure 13. Plastic hinges developed in the original and in the retrofitted frames

The left side of Figs. 13a-c show the cumulative plastic hinges developed on the original frame The reduction on the structural response of the rehabilitated frame can be observed in those figures at the right.

CONCLUSIONS

A performance-based design approach was proposed for the rehabilitation of buildings with energy dissipating devices However, it can also be used for alternative rehabilitation solutions, such as bracing element systems. The approach also may be seen as an *algorithm* useful for converging to a first approximation of a system to be analyzed according to the time history approach recommended in NEHRP Guidelines (FEMA 273, 1997).

The acceptance criteria proposed for the design are based on controlling the peak interstory drifts of the rehabilitated structure and the maximum ductility demands of the dissipators. The plastic hinge rotation demands of the structural members are also verified to be within tolerable limits.

Analyzing equivalent SDOF systems instead of using the whole MDOF model leads to smaller computer processing time. It is remarked that once that the SDOF models are analyzed, the final solution regarding the MDOF system can be obtained in one or two iterations. This makes attractive the proposed algorithm.

The algorithm proposed was applied successfully for the rehabilitation of a ten-story three-bay reinforced concrete frame.

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CURSO INTERNACIONAL DE INGENIERIA SISMICA

DISEÑO SISMICO DE UN EDIFICIO DE CONCRETO REFORZADO

Sergio M. Alcocer

1. DESCRIPCION DEL EDIFICIO
2. ESTRUCTURACION
3. DIMENSIONAMIENTO (DISEÑO PRELIMINAR)
4. ANALISIS ESTRUCTURAL
5. DISEÑO
6. INTERACCION SUELO-ESTRUCTURA

1. DESCRIPCION DEL EDIFICIO

- Centro de Salud: Grupo A
- Seis pisos más un sótano
- Altura de entrepiso: sótano y planta baja: 4.5 m
  resto: 3.6 m
- Area total construída: 5440 m2 (7 niveles)
- Zona III, D.F.
- Ver planta anexa

2. ESTRUCTURACION

a) Material

Concreto reforzado: economia

+ resistencia de diseño del concreto a la compresión: $f_c' = 300\ kg/cm^2$: clase 1
+ resistencia de fluencia del acero: $f_y = 4200\ kg/cm^2$

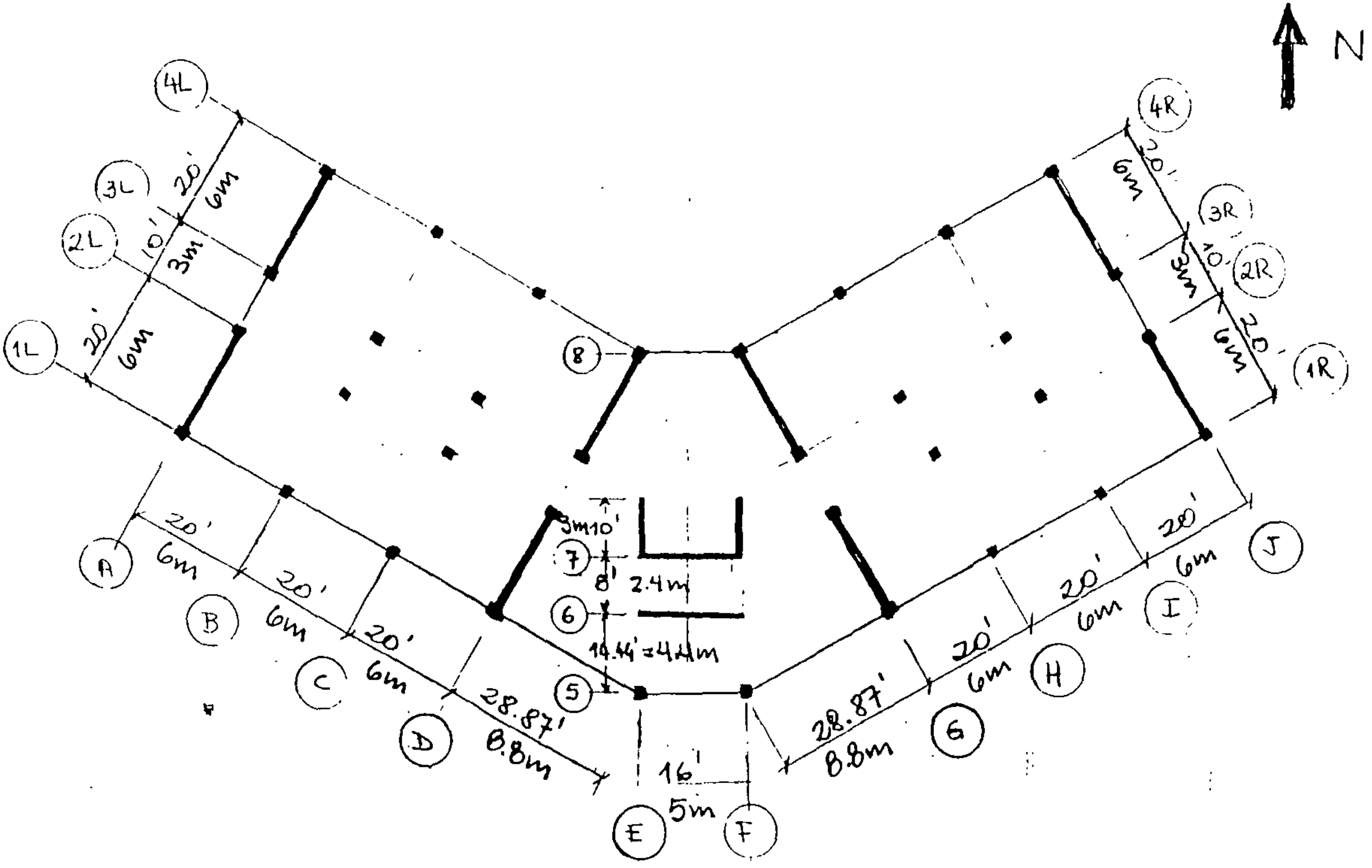
b) Estructuración

- marcos dúctiles: RDF, Q = 4
- muros, Q = 3
- marcos-muros, Q = 3 **

Sistema resistente de fuerzas laterales:

- muros acoplados en la dirección NS
- núcleo de elevadores y escaleras en la dirección EW

  **** Los muros acoplados controlan desplazamientos de las alas



PLANTA DEL EDIFICIO

Sistema resistente de cargas gravitacionales

Marco

- losa en dos direcciones
- vigas interiores y de fachada
- columnas

Cajón de cimentación

- losa y contratrabes
- muro perimetral de contención

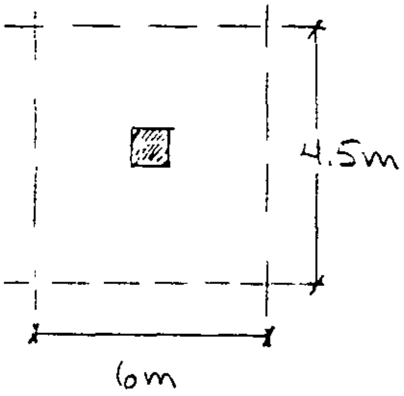
Muros divisorios

- no formarán parte del sistema resistente a sismo

3. DIMENSIONAMIENTO (DISEÑO PRELIMINAR)

a) COLUMNAS

PARA UNA COLUMNA INTERIOR ; CARGAS VIVA MÁXIMA
MUERTA



CV* $w_m = 170\ kg/m^2$ piso
$100\ kg/m^2$ azotea
* según RDF Art 199

CM $w = 650\ kg/m^2$ piso
$500\ kg/m^2$ azotea

Carga Axial Ultima en el sótano:

$$P_u = 1.4 \times 6 \times 4.5 \left[\left(0.65 \times \underbrace{\overset{CM}{6}}_{6\,pisos} + 0.5 \right) + \left(0.17 \times \overset{CV}{6} + 0.10 \right) \right]$$

$$P_u = 209\ ton$$

Nota La carga viva no se reduce pues $A_T = 6 \times 4.5 = 27 < 36\ m^2$

Si aceptamos $\sigma_{perm} = 100\ kg/cm^2$ $(1000\ ton/m^2)$

$$A = \frac{P_u}{\sigma_{perm}} = \frac{209}{1000} = 0.21\ m^2$$

$$\Rightarrow \quad h = \sqrt{A} = 0.46\ m$$

Usar columnas 45×45 cm | Sección constante en la altura

b) LOSA

De acuerdo a NTC 4.3.3 (Losas apoyadas en su perimetro)

$$h_{min} = \frac{\text{perimetro tablero}}{300} \times 0.034 \sqrt[4]{f_s w}$$

si $f_s > 2000$ kg/cm²
y/o $w > 380$ kg/m²

Sea el tablero entre los ejes: 1L-2L-A-B
+ 2 lados continuos de 6m
+ 2 lados discontinuos de 6m

$$\text{Perimetro} = 2 \times 6m + 2 \times 6m \times 1.25$$

↳ por ser discontinuos

$$= 27m$$

Si $f_s = 0.6 f_y = 0.6 \times 4200$
y $w = 720$ kg/m² (CM + CV media)

$$h_{min} = \frac{2700}{300} \times 0.034 \sqrt{0.6 \times 4200 \times 720}$$

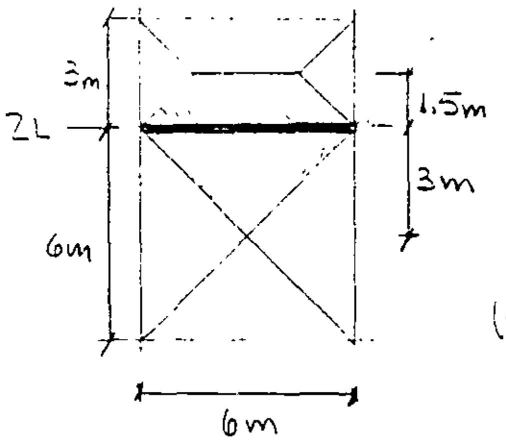
$$h_{min} = 11 \text{ cm}$$

Puesto que en los ejes D, E, F, G no existe acción de marco usamos

$$\boxed{h = 18 \text{cm}}$$

c) VIGAS

Puesto que se ha supuesto que la resistencia lateral está proporcionada por los muros, verifiquemos que una viga de 35×50 cm de sección pueda reforzarse para carga vertical

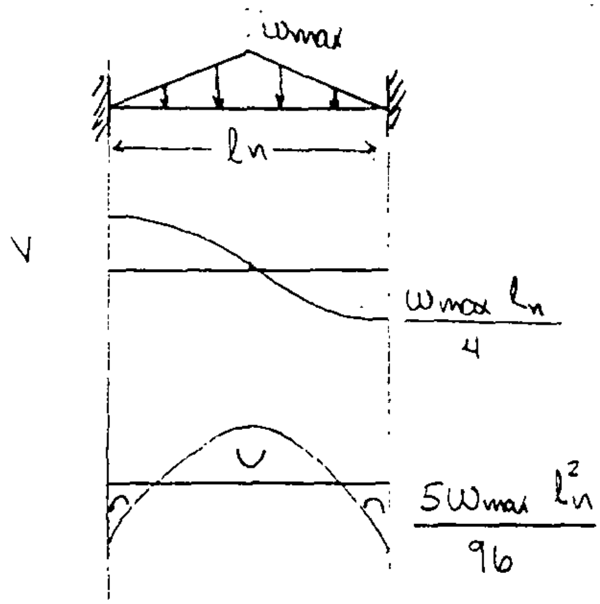


$A_T = \frac{6^2}{4} + \frac{6+3}{2} 1.5 = 15.75 m^2$

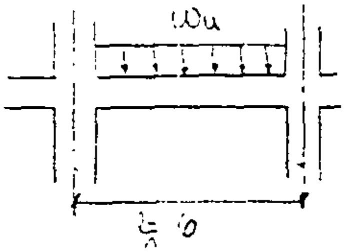
Losa CM = 500 kg/m²
(sin considerar columnas, muros concreto, vigas)
y CV = 170 kg/m²

Carga trapecial : tratada como uniforme

Carga triangular: transformar a carga equivalente uniforme que produzca los mismos momentos de empotramiento



$$\frac{5 w_{max} l_n^2}{96} = \underline{\underline{\frac{5}{8}}} \frac{w_{max} l_n^2}{12}$$



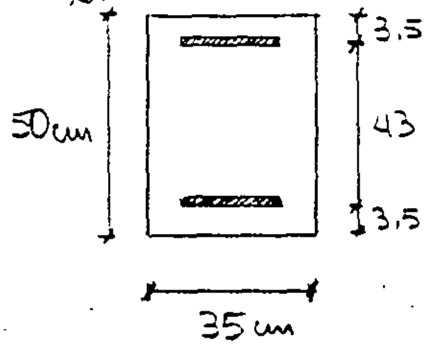
$$\omega_u = 1.4 \Big(\overset{CM}{500 \times 3 \times \underline{\underline{\frac{5}{8}}} + 500 \times 1.5} +$$

$$+ \underset{\substack{\downarrow \\ \text{viga}}}{0.35} (0.5 - 0.18) \times \underset{W_{\text{colindan}}}{2200} + 170 \times 3 \times 5/8 +$$

$$+ 170 \times 1.5 \Big) = 3510.7 \ \text{kg}/\text{m}$$

$$\text{Si} \quad M_u = \frac{w_u l_n^2}{10} = \frac{3511 (6 - 0.45)^2}{10} = 10{,}815 \ \text{kg} \cdot \text{m}$$

Si



$$d = 46.5 \text{cm}$$

$$M_u = M_R = F_R \, \underline{0.9d} \, A_s f_y$$

$$A_{s\,req} = \frac{M_R}{0.9 \times \underline{0.9 \times 46.5} \times 4200}$$

$$A_{s\,req} = 6.8 \ \text{cm}^2$$

$$\rho_{req} = \frac{A_{s\,req}}{bd} = 0.0042$$

Para una sección balanceada (Ecuación 2.2, NTC)

$$\rho_{bal} = \frac{f''_c}{f_y} \; \frac{4800}{f_y + 6000}$$

donde $f^*_c = 0.8 f'_c = 0.8 \cdot 300 = 240 kg/cm^2$

y $f''_c = 0.85 f^*_c = 204 kg/cm^2$

$$\Rightarrow \rho_{bal} = \frac{204}{4200} - \frac{4800}{4200+6000} = 0.0229$$

Pero, según NTC 2.1.2.b,

$$\rho_{max} = 0.75 \rho_{bal} = 0.0171 > \rho_{req} \quad ✓✓$$

Posiblemente una viga más pequeña sería adecuada. Sin embargo, los efectos sísmicos no se han considerado aún. El peralte, además, parece adecuado para un claro de 6m. $\left(\frac{L}{h} = \frac{600}{50} = 12 ✓ \right)$

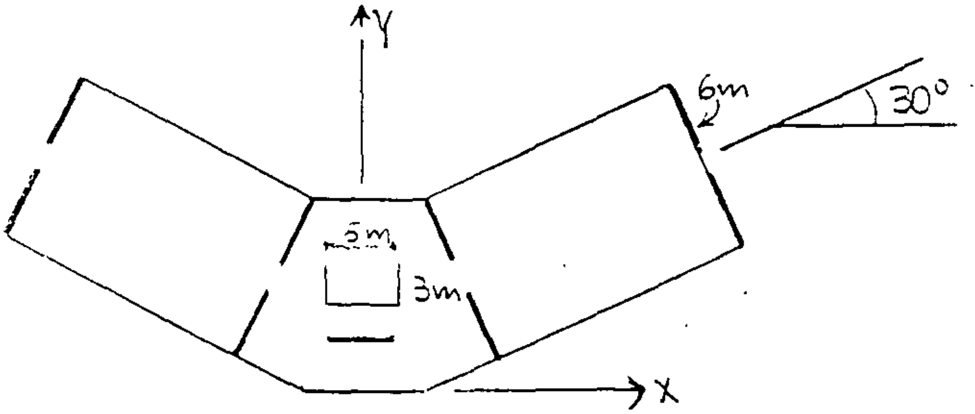
$\Rightarrow$ Usar 35×50 cm | para todas las vigas de las alas.

Nota: Se propuso 35cm. para que la relación entre los anchos de viga y columna (i.e., 35/45=0.78) sea 75%, aprox.

d) MUROS

Se dimensionan los muros para resistir todo el cortante sísmico.

Croquis



Las fuerzas cortantes sísmicas totales son:

$V_X = 237$ ton

$V_Y = 199$ ton

Criterio de Diseño: el esfuerzo cortante promedio en el concreto será menor que $0,5\sqrt{f^*_c}$

$\downarrow$

tomado de la Ec. 2.18 ($V_{cR} = \underline{\underline{0.5}} F_R b d \underline{\underline{\sqrt{f^*_c}}}$) para $\rho \geq 0.01$

Criterio de Análisis: los muros "inclinados" participan en las resistencias sobre X y Y en función de su área multiplicada por $\sin^2 30°$ y $\cos^2 30°$, resp.

Dirección X (la otra dirección no es crítica, por inspección)

$$A_{req} = \frac{1.1 \times 237000 \text{ kg}}{\underset{\uparrow F_R}{0.80}} \frac{1}{0.5\sqrt{240}} = 42070 \text{ cm}^2 = 4.2 \text{ m}^2$$

Si $t = 20$ cm

$$A_{muro} = 8 \cdot (6 \times 0.20 \times \sin^2 30°) + 2(5 \times 0.20)$$

$$= 4.4 \text{ m}^2$$ ✓

RESUMEN

Secciones constantes en la altura

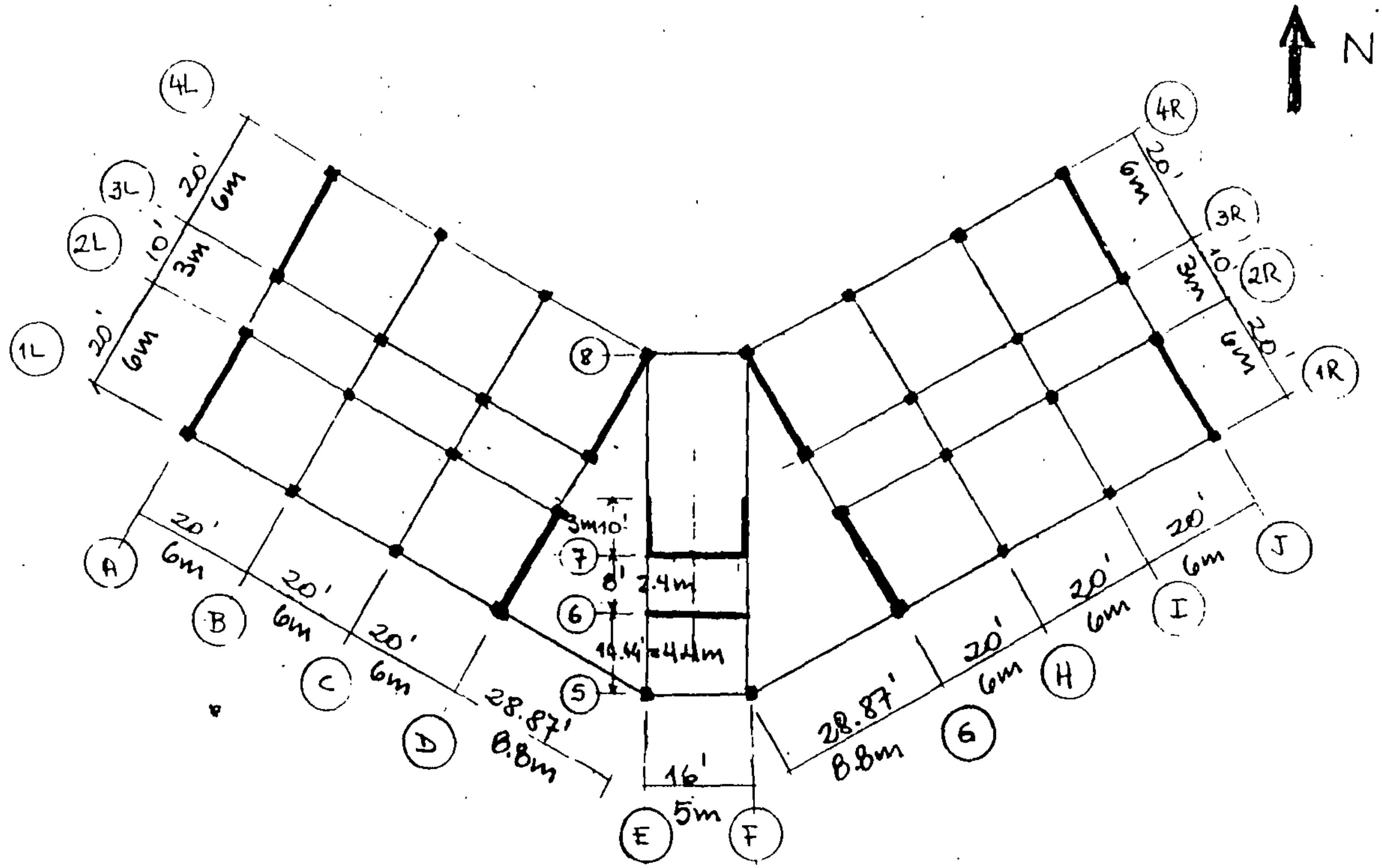
COLUMNAS : 45 cm × 45 cm

LOSA : 18 cm (t: espesor)

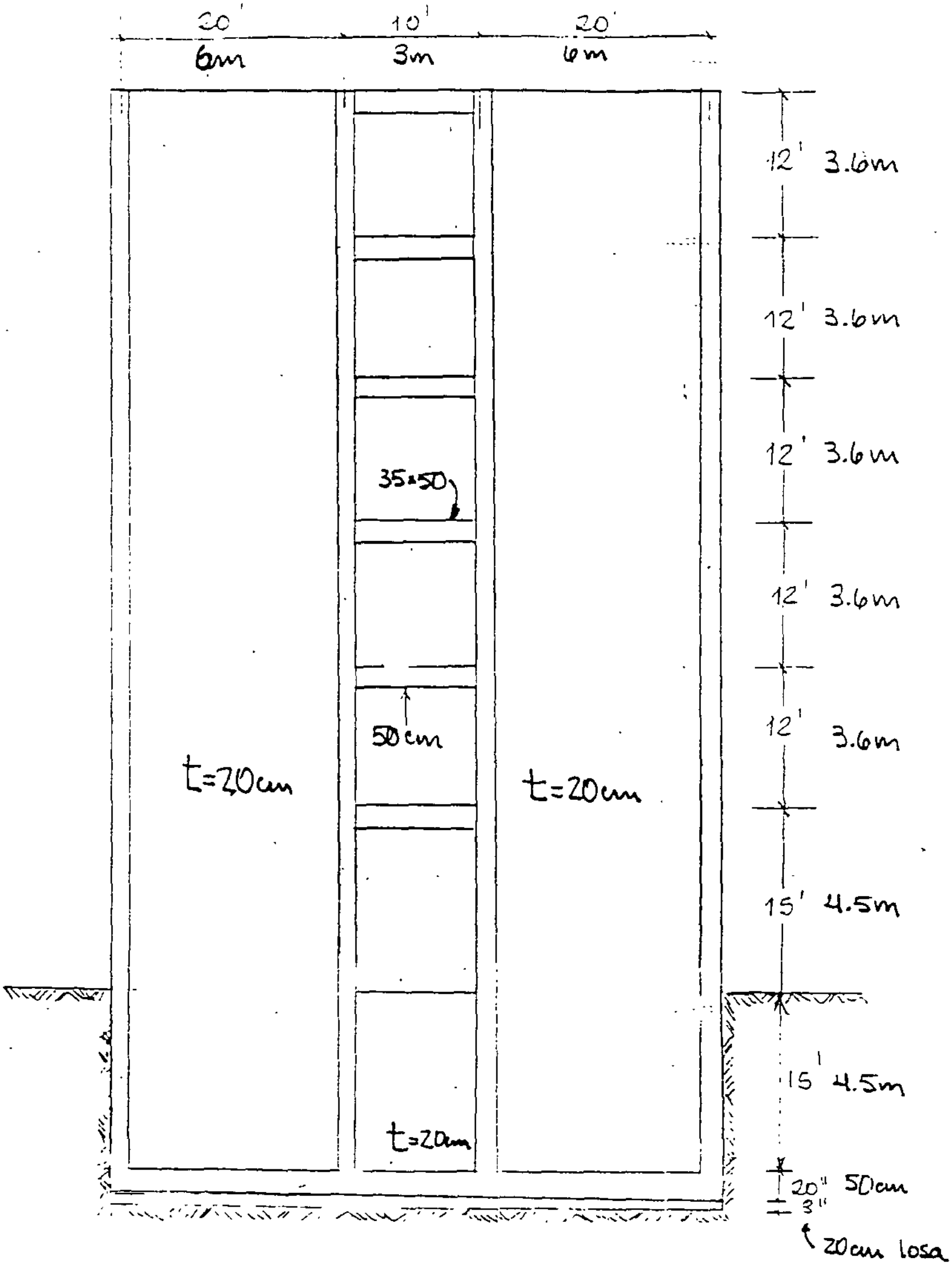
VIGAS : 35 cm × 50 cm (también contratrabes)

MUROS : 20 cm (t. espesor)

LOSA CIMENTACIÓN . 20 cm (espesor)



PLANTA DEL EDIFICIO



ELEVACIÓN DE MUROS ACOPLADOS

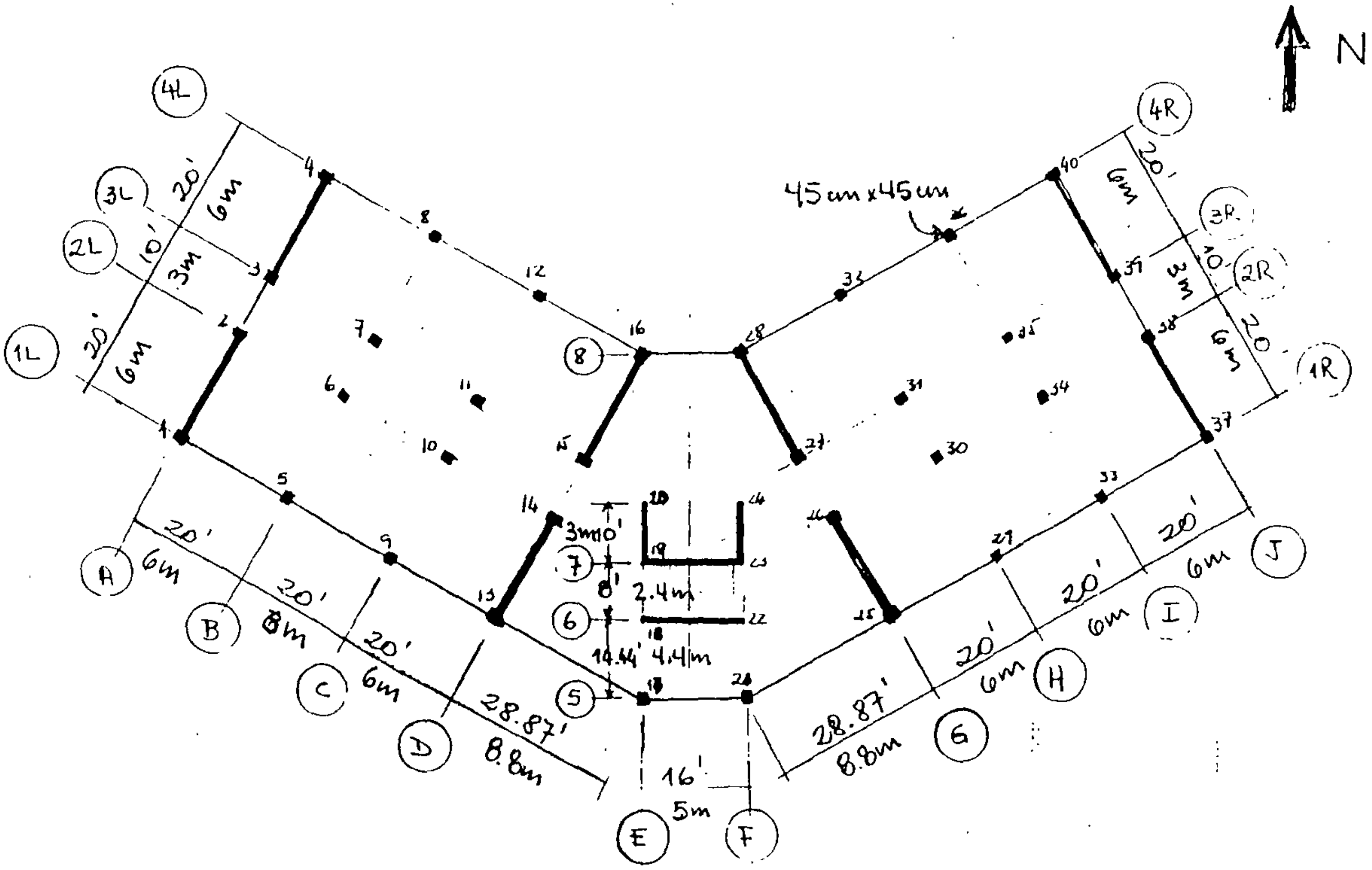
4. ANALISIS ESTRUCTURAL

Hipótesis

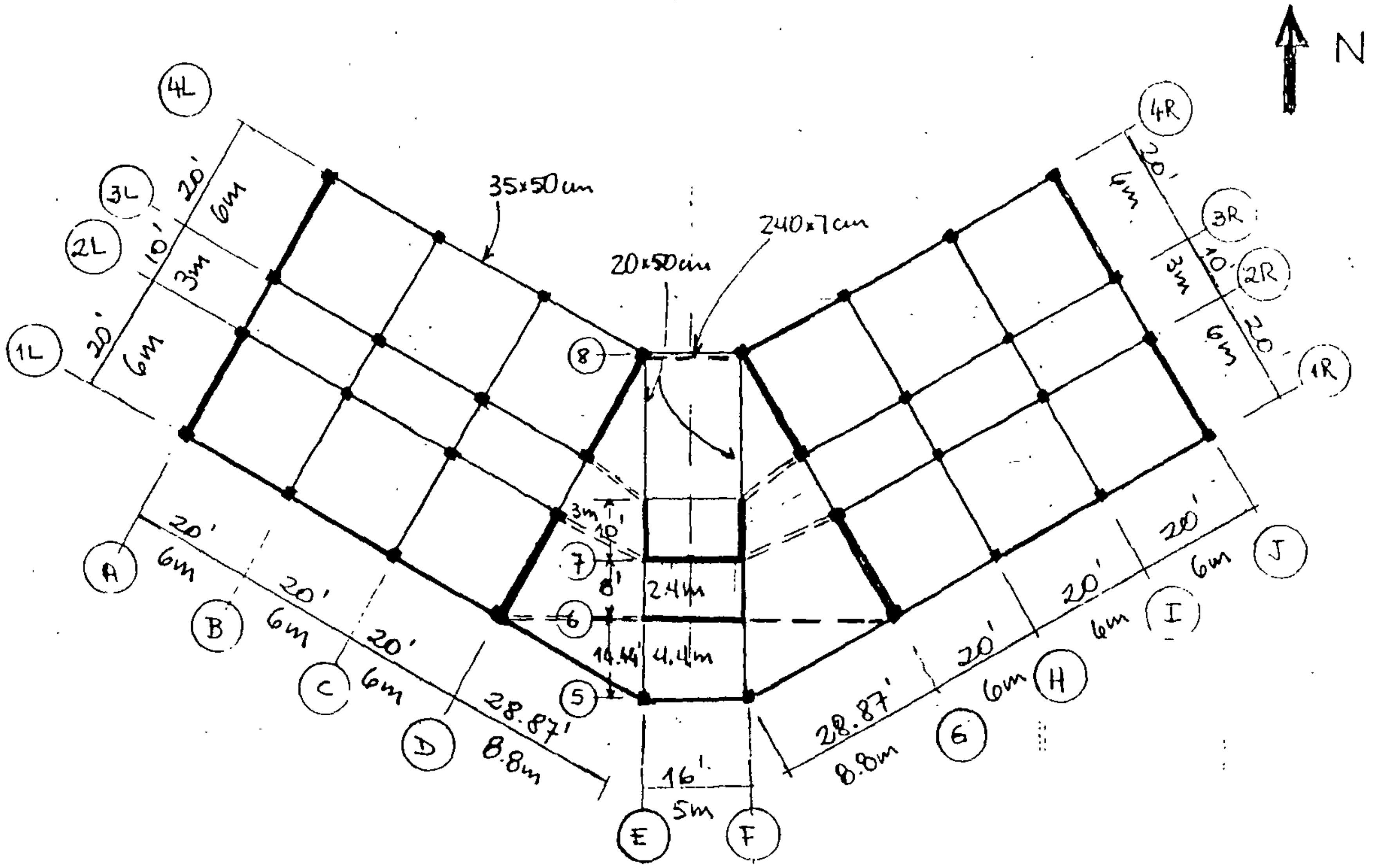
- análisis elástico
- losa: diafragma rígido, aunque la forma del edificio sugiere "aleteo" del edificio
- se supone que el cajón de cimentación es rígido
- los elementos no estructurales (muros divisorios) no se consideran en el análisis

El edificio se modeló y analizó como estructura tridimensional usando el programa SUPER-ETABS

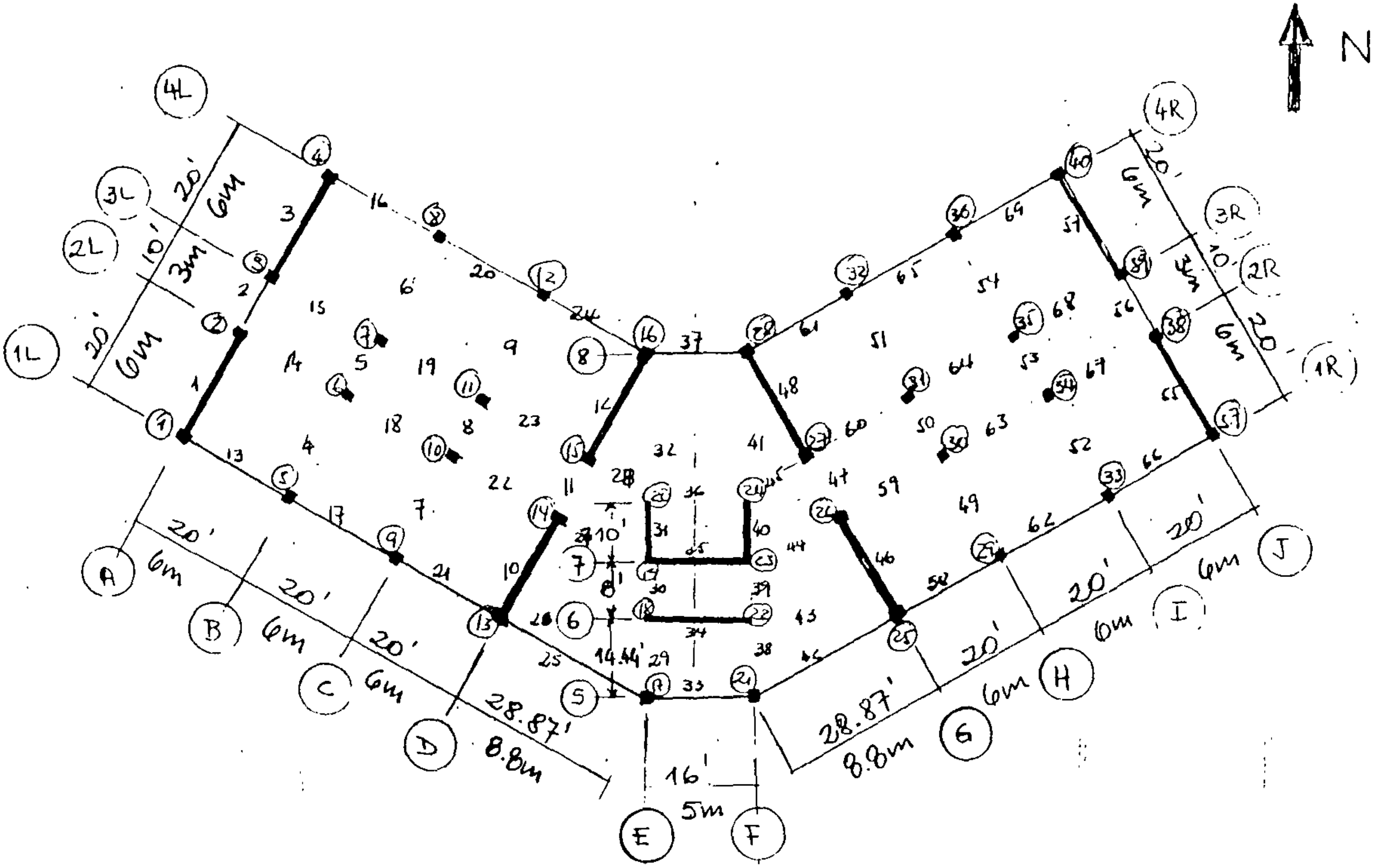
40 líneas de columna
69 crujías
102 elementos muro (paneles)
12 "contravientos flexibles" para estimar la distorsión ($AE/L=6.4\ kg/cm$)



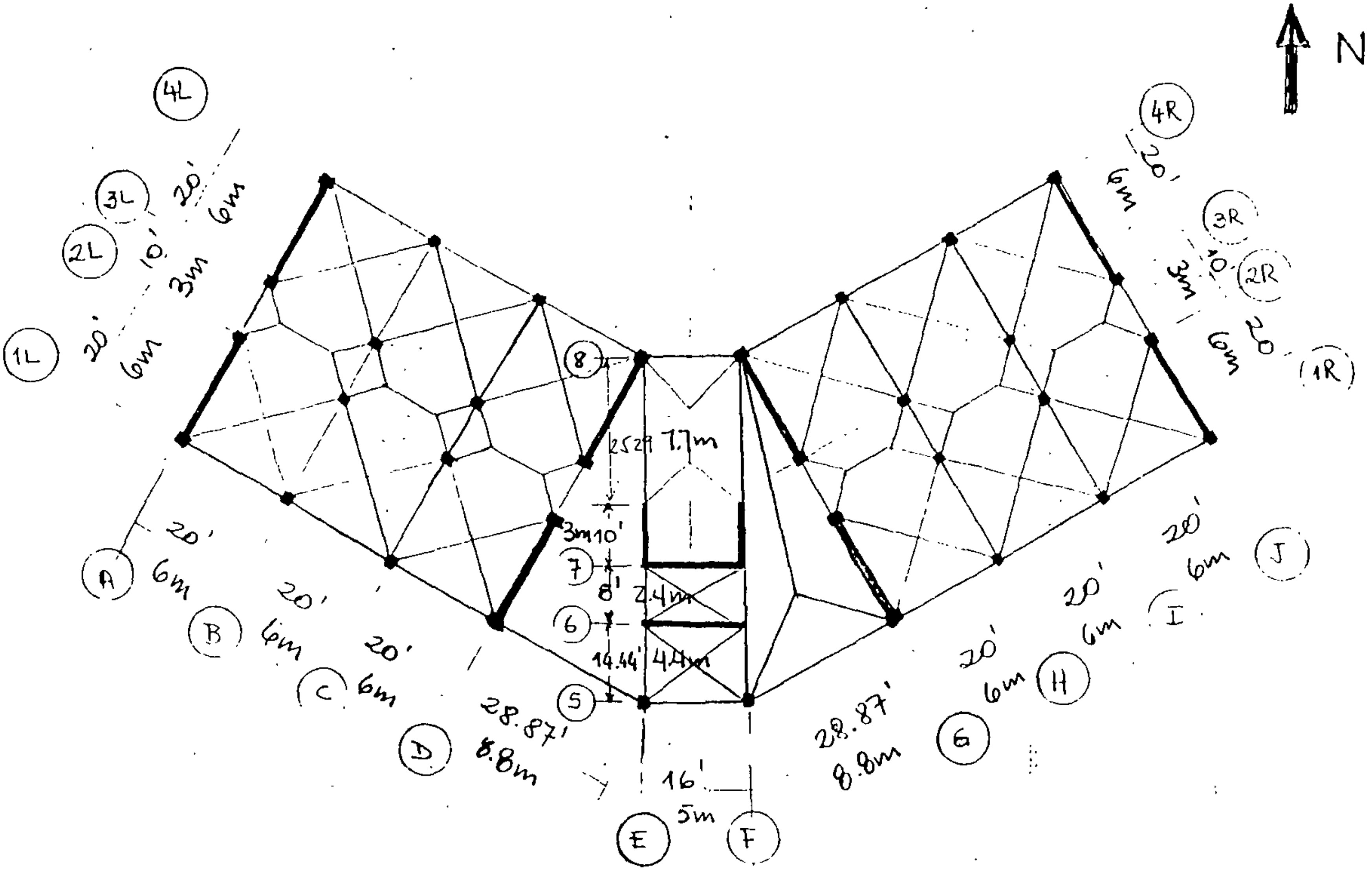
NUMERACIÓN DE LÍNEAS DE COLUMNAS.



DISTRIBUCIÓN DE LAS VIGAS



NUMERACIÓN DE CRUJIÀS.



AREAS TRIBUTARIAS

CARGAS VERTICALES

a) ANÁLISIS ESTÁTICO

- altura total de la estructura: $2\times 4.5\,m + 5\times 3.6\,m$
$= 27\,m < 60\,m$

- $$\frac{V}{W} = \frac{c}{Q'}$$

donde: $c = 1.5\ (0.4) = 0.6$

↳ c para Zona III

↳ factor de importancia de la estructura: Grupo A.

$Q = 3$ puesto que no se satisface que:

"en cada entrepiso los marcos son capaces de resistir, sin contar muros ni contravientos, cuando menos 50% de la fuerza sísmica actuante" (NTCSismo)

Asimismo, en 4.5.2 de las NTC-Concreto, se señala que:

"en el diseño por sismo de los muros a que se refiere esta sección y que resistan la totalidad de las fuerzas laterales, se usará $Q' = 3$"

Pero $Q' = \underset{\text{regularidad}}{\text{Factor}}\quad Q$

El edificio no satisface las condiciones de regularidad:
1.- Simetría
10.- Torsión

$\Rightarrow \quad Q' = 0.8\,Q = 2.4$

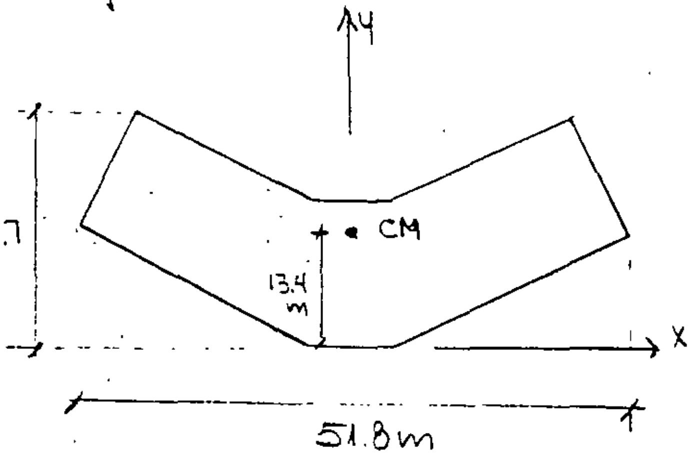
Por tanto, $\frac{V}{W} = \frac{0.6}{2.4}$

$$\frac{V}{W} = 0.25$$

* Los desplazamientos de entrepiso se multiplicarán por Q=3. Se verificará que. desplazamiento / altura entrepiso < 0.012
| porque los muros divisorios ↑
| están separados de la estructura

Las fuerzas laterales se distribuyen en la altura del edificio. Estas fuerzas actuarán en el centro de masa del nivel (y las fuerzas cortantes en el centro de cortante).

Para este edificio:

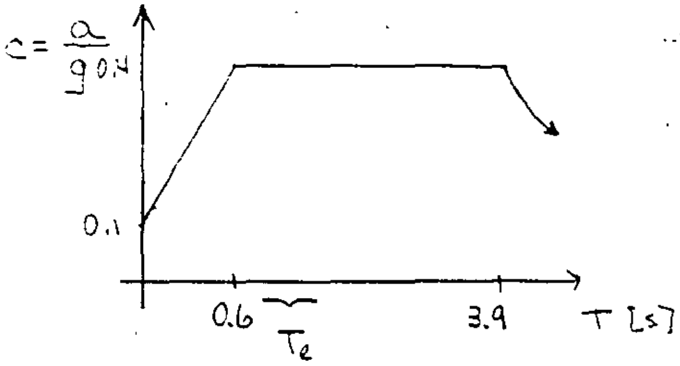


* importante considerar torsión en el análisis (J. Damy)

b) ANÁLISIS DINÁMICO

- debido a la asimetría

- espectro de diseño Zona III (Zona de Lago)



$$T_e = 0.13 \sim 0.14 N \approx 0.8 s$$

- incluir $T_i \geq 0.4s$

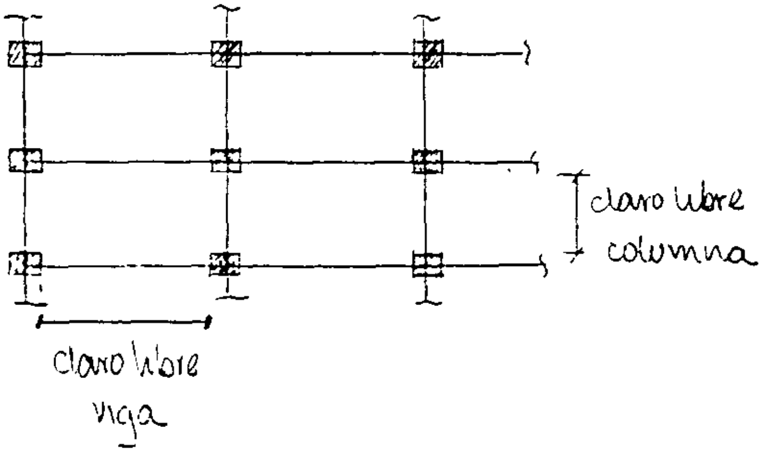
- usar espectro reducido : $Q' = 2.4$

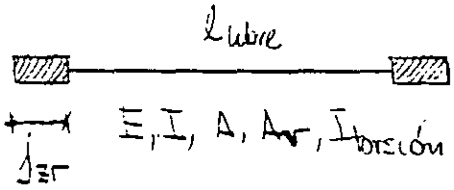
- combinar respuestas : $S = \sqrt{\sum_i S_i^2}$

+ si los periodos de modos naturales difieren en menos de 10% entre sí : CQC (Complete Quadratic Combination - ETABS)

NOTA : las respuestas modales combinadas son las acciones, desplazamientos, y elementos mecánicos a estudiar. NO usar una respuesta combinada S para obtener otro parámetro.

c) IDEALIZACIÓN DE LA ESTRUCTURA





Si las propiedades de las secciones (A, I) son en base a secciones no agrietadas, usar $l_{zr} = \frac{1}{3}$ ancho nudo

Si se usan propiedades agrietadas, $l_{zr} = \frac{1}{2}$ ancho nudo.

Agrietamiento: vigas 0.4 I_g
columnas 0.8 I_g
muros 0.25 I_g

Para vigas

"L" : $I \simeq 1.5\, I_{rectangular}$

"T" : $I \simeq 2\, I_{rectangular}$

Para el análisis de la estructura, se consideraron las columnas y muros de la PB y 1^er^ piso como agrietados.

d) ANÁLISIS: se usó el Uniform Building Code 1985

- Pesos: Azotea : 579 ton
  Piso : 728 ton

Total : 4219 ton (no considera el peso del sótano)

Para V_x, $c = 0.056\,W$ | para roca
V_y, $c = 0.047\,W$

Piso	Fuerzas [ton]	
	X- EW	Y- NS
5	55.6	46.6
4	58.7	49.2
3	47.5	39.9
2	36.3	30.5
1	25.1	21.1
PB	14.4	12.1

- Verificación del modelo matemático.

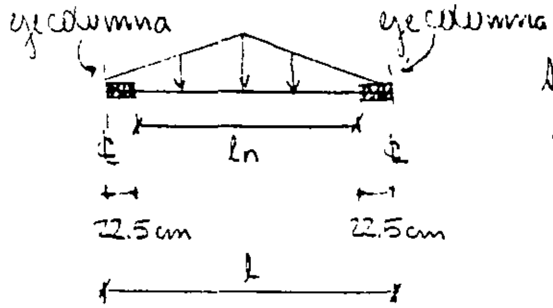
1) Simetría: cargas simétricas (verticales, sismo etc) produjeron respuesta simétrica
- "Chequeo" con carga vertical

2) Cargas verticales

$$(\Sigma P_u)_{\text{análisis}} = (W_u)_{\text{espacio}} \quad (1)$$

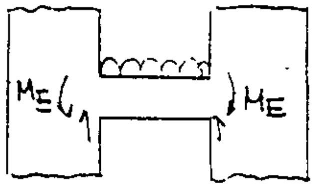
pero, ✓ < ✓ de la salida.

La explicación es que los cortantes de empotramiento se calcularon usando el claro libre

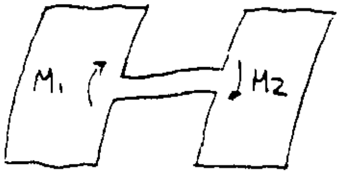


Si se usa l en lugar de l_n, se verifica la ec (1).

3) otro efecto : ante carga vertical, los M de las vigas de acoplamiento son diferentes de M_E



Empotramiento



Ante carga vertical

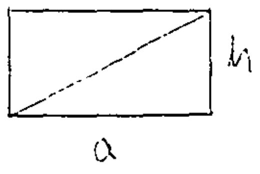
Resultados del Análisis ante sismo.

- Análisis Dinámico · periodos

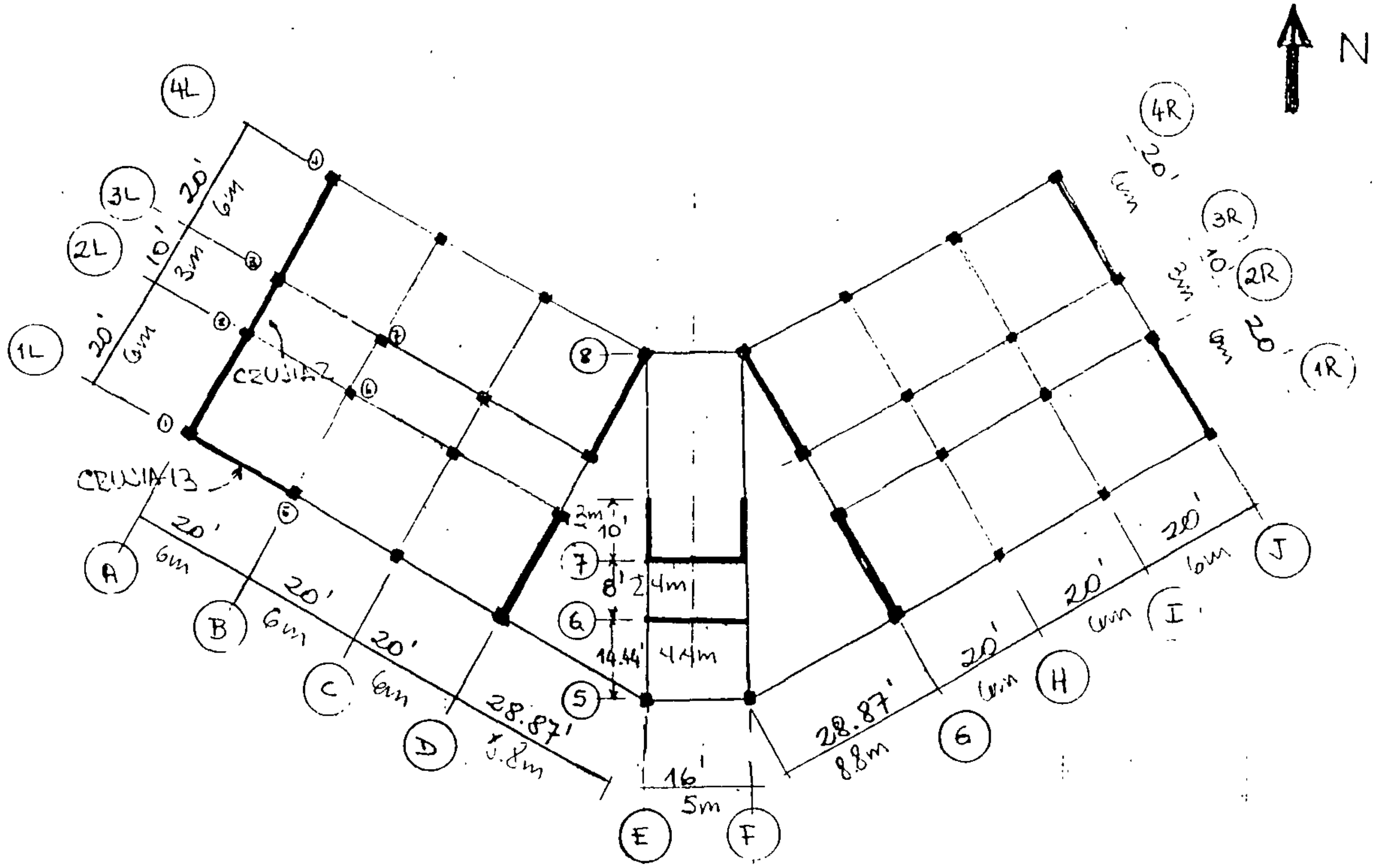
$T_x = 0.76 s$
$T_y = 0.61 s$
$T_\theta = 0.28 s$
| masa modal efectiva 70% RCSC

- Contravientos para medir distorsiones
  - crujías 2 y 13 : para medir torsión
  - distorsión $= 0.1N \left(\frac{a}{h} + \frac{h}{a} \right)$

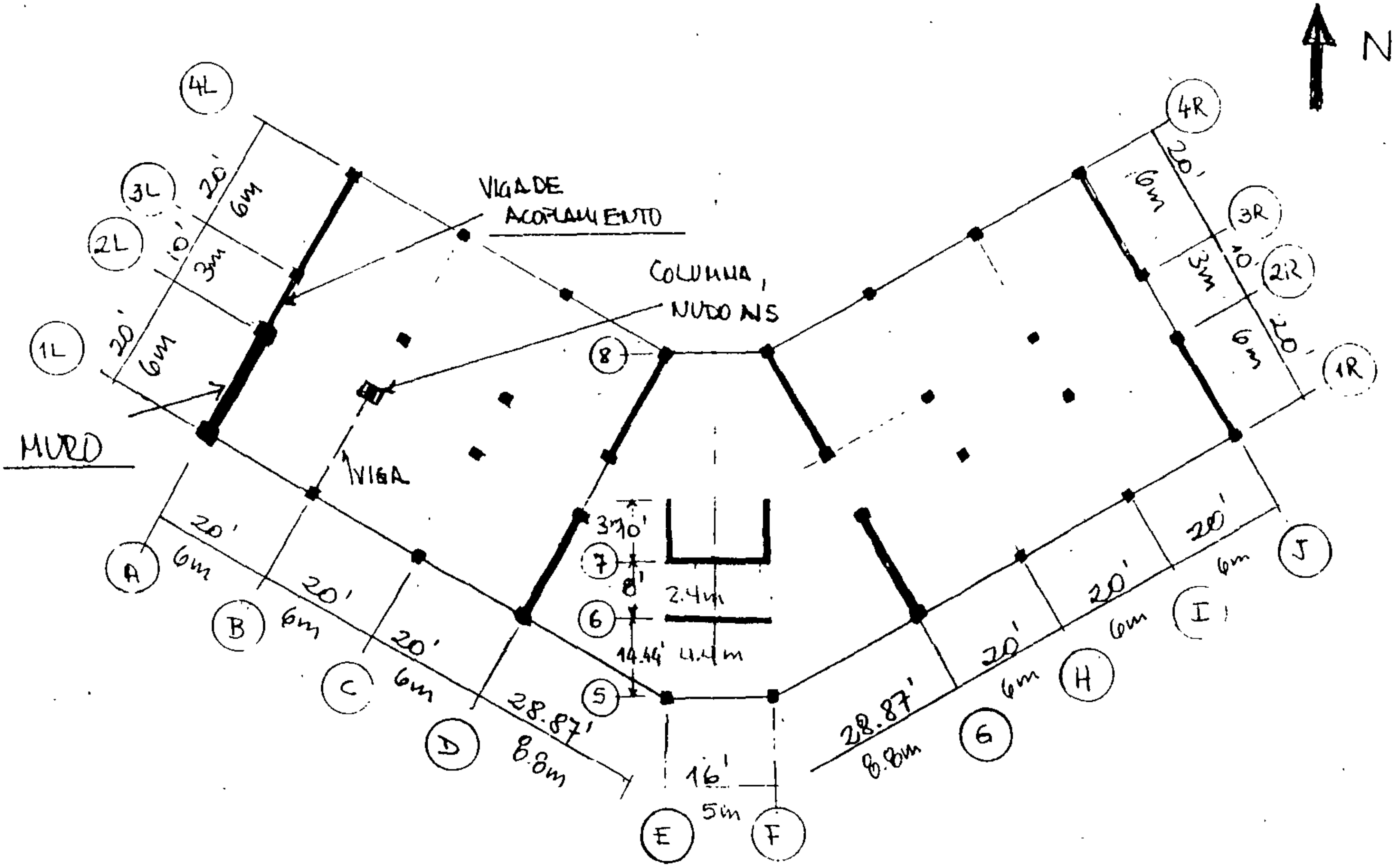
↑ fuerza normal · incluye superposición modal



+ permite eliminar el cabeceo (cimentación flexible)
+ realmente mide los desplazamientos en los extremos de las alas versus el desplazamiento del piso registrado en el CM
+ la máxima distorsión = 0.002.



CONTRAVIENTOS PARA ESTIMAR LA DISTORSIÓN



ELEMENTOS DISEÑADOS

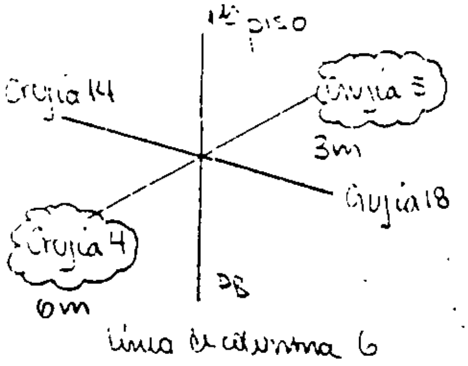
5- DISEÑO

Se diseñarán uno de los siguientes elementos

a) Viga
b) Columna.
c) Junta Viga-columna
d) Viga de acoplamiento
e) Muro

NOTA: Se garantizarán las dimensiones en la construcción

a) VIGAS



Vigas: 35×50 cm d = 46.5 cm

Columna 45×45 cm

a.1) Determinar As para vigas

$$\rho_{min} = \frac{0.7\sqrt{f'c}}{fy} = \frac{0.7\sqrt{300}}{4200} = 0.00289^{*}$$

$$\rho_{max} = 0.75\rho_{bal} = 0.0171$$ (ver Dimensionamiento)

* en cada lecho

As_{min} = 2#4 continuas en el nudo

Crujía 4. ✓ $M_u^- = 1660.6$ ton cm $\Rightarrow \rho = 0.0064 \Rightarrow A_s = 10.5$ cm²

4#6 , $A_s = 11.36$ cm²

$M_u^+ = 554.7$ ton cm $\Rightarrow \rho = 0.0022 < \rho_{min} \Rightarrow A_s = 4.7$

Crujía 5: $M_u^- = 1210.8$ ton cm $\Rightarrow \rho = 0.0047 \Rightarrow A_s = 7.65$ cm²

✓ $M_u^+ = 871.8$ ton cm $\Rightarrow \rho = 0.0024 \Rightarrow A_s = 3.51$ cm²

3#5 , $A_s = 6.0$ cm²

Si revisamos, usando ec. 2.9 NTC-Concreto

$$M_R = F_R[(A_s - A_s')f_y(d - a/2) + A_s' f_y (d - d')]$$

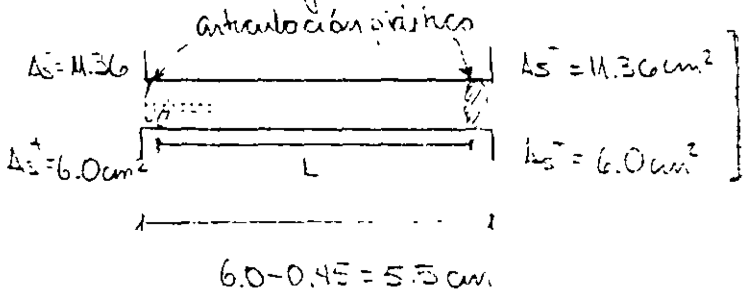
$$= 0.9[(11.36 - 6.0)4200(46.5 - 3.15/2) + 6.0 \times 4200(46.5 - 3.5)]$$

donde $a = \dfrac{(A_s - A_s')f_y}{f''_c b} = \dfrac{(11.36 - 6)4200}{204 \cdot 35} = 3.15$

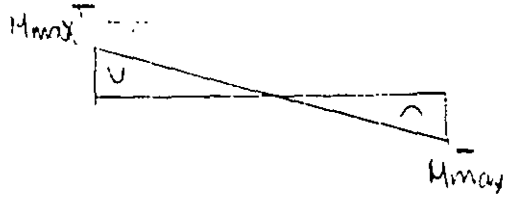
$M_R^- = 1885.4$ ton cm $> M_u^-$ para la crujía 4.

a.2) Refuerzo de cortante

Se pensamos que el acero obtenido se mantiene en las secciones críticas de la crujía 4:



Se satisface que $M_R^+ \geq 0.5 M_R^-$



Según 3.2.4 de NTC podemos diseñar para fuerza cortante:

+ por capacidad : sin factores de reducción
esfuerzo acero = 1.25 fy
carga viva afectada por Fc

$$V_{max} = \frac{M^+_{max} + M^-_{max}}{L} + \frac{wL}{2} \approx \frac{\alpha(M^+_{ij} + M^-_{ij})}{L} + \frac{wL}{2}$$

$$M_n^- = \frac{M_e^-}{F_R} = \frac{1885.4}{0.9} = 2095 \text{ ton·cm}$$

$$M_n^+ = 1127 \text{ ton·cm}$$

$w = 2387$ kg/m (obtenido en forma similar al prediseño)

$$V_{max} = \frac{(2095 + 1127)\,1.25}{600} + \frac{2.4 \times 6}{2}$$

$$V_{max} = 6.7 + 7.2 = 14.0 \text{ ton}$$

$$V_n = \frac{V_{max}}{1}_{F_R = 1} = 14.0 \text{ ton}$$

+ de Vanálisis, con Fr = 0.6 (en lugar de 0.8)

$$V_{análisis} = 11.7 \text{ ton}$$

$$V_n = \frac{V_{análisis}}{0.6} = 19.5 \text{ ton}$$

⇒ diseñar para $V_n = 19.5$ ton

Puesto que $V_{sismo} < 0.5\, V_u \sim (11.5t)$

se considera la contribución del concreto a cortante.

Puesto que $p < 0.01$, usar ec. 2.17:

$$V_{CR} = \underset{\downarrow}{F_R}\, bd\,(0.2 + 30p)\sqrt{f^*_c}$$

$$= 0.6 \quad 35 \times 46.5(0.2 + 30 \cdot 0.0064)\sqrt{240}$$

$$= 5930 \text{ kg} < 1.5 \times \underset{\hookrightarrow F_R}{0.6}\, bd\sqrt{f^*_c} = 22\,690 \text{ kg}$$

acero toma $\quad V_u - V_{CR} = 11.7 - 5.9 = 5.8 \text{ ton}$

$$S = \frac{F_R A_v f_y d}{V_u - V_{CR}} \qquad \text{ec. } 2.21$$

E#3
cerrados

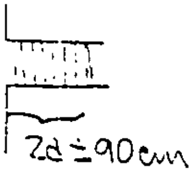
$$S = \frac{0.6 \cdot 2 \cdot 0.71 \times 4200 \cdot 46.5}{5800} = 29 \text{cm} \,!$$

$$S_{max} = \frac{0.6 \times 2 \cdot 0.71 \cdot 4200}{3.5 \times 35} = 29 \text{ cm}, \; y \; S_{max} = 0.5d = 20 \text{cm}$$

$$\boxed{E\#3, \text{ cerrados } \; s = 20 \text{cm}}$$

a.3) Esfuerzo por confinamiento.

+ extremo: 2d a partir cara columna

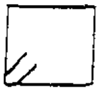


1er estribo a 5cm de la cara

Sección 5.2.3

$$S \leq \begin{cases} 0.25d = & 11\text{ cm} \\ 8 d_{b_{min}} = & 12.5\text{ cm} \quad (\#5) \\ 24 d_{b_{estribo}} = & 23\text{ cm} \\ 30\text{ cm} & \end{cases}$$

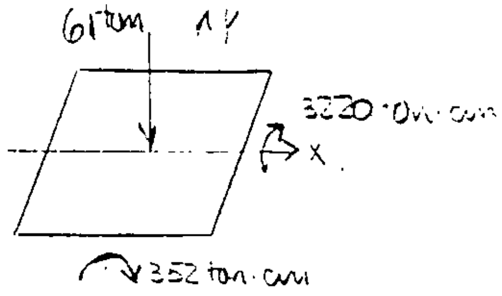
usar s=10cm



- dobleces 135°
- extensión 10 d_b (9.5cm) !

Fuera de la zona de la articulación plástica: $S_{max} \leq 0.5 d = 23$ cm
usar s= 20 cm (O.K.)

b) COLUMNA



b.1) Diseño por flexión

$M_{ux} = 352$ ton·cm

$M_{uy} = 3220$ ton·cm

$e_x = 6$ cm
$e_y = 53$ cm $\Big\} > 0.05h = 2.25$ cm ✓ ($\uparrow$ e min)

Según 5.3 (NTC-Concreto) miembro a flexocompresión si

$$P_u > A_g f'/10$$

$$\frac{A_g f'_c}{10} = \frac{45^2 \cdot 300}{10} = 60.8 \text{ ton}$$ (está en el límite de ser considerada a flexión)

Para fines del ejemplo ⇒ flexocompresión

Usaremos las gráficas para diseñar columnas de concreto reforzado (Serie 428 Instituto de Ingeniería)

$$R_x = \frac{352000}{\underset{\hookrightarrow F_R}{0.8} \cdot 45^3 \cdot 204} = 0.024 \qquad \left(R = \frac{M}{F_R b h^2 f''_c} \right)$$

$$R_x / R_y = 0.11$$

$$R_y = \frac{3220000}{0.8 \cdot 45^3 \cdot 204} = 0.217$$

$$K = \frac{P_u}{F_R b h f''_c} = \frac{61000}{0.8 \cdot 45^2 \cdot 204} = 0.185$$

Peralte efectivo : $d = h - r = 45 - r = 40$cm

$r = 3+2 = 5$ cm

$\Rightarrow$ $d/b = 40/45 = 0.89$ usar 0.9

Para $e_x/e_y = 0.5$, Figura 42 $q = 0.7$

$e_x/e_y = 0$, Figura 10 $q = 0.5$

interpolando para $e_x/e_y = 0.11$, $q = 0.54$

Pero $q = \frac{A_s}{bh} \frac{f_y}{f''_c}$

$$A_s = q bh \frac{f''_c}{f_y} = 0.54 \times 45 \times 45 \frac{204}{4200}$$

$A_s = 53.1$ cm^2

Usar 12 #8 $A_s = 61.2$ cm^2

$\rho = 61.2/45^2 = 0.03 < \rho_{max} = 0.04$

$\rho_{min} = 0.01$

Para miembros verticales se satisface que:

- $h_{min} \geq 30$cm
- $A_g > P_u/0.5 f'_c$
- $b/h \nless 0.4$
- $H'/h = 310/45 = 6.9 < 15$

Para asegurar un comportamiento "viga débil - columna fuerte"

$$\frac{\Sigma M_{col}}{\Sigma M_{vigas}} \geq 1.5$$ donde los momentos se refieren a momentos resistentes reducidos al centro del nudo

Para Mc columna : $A_s = 61.2\ cm^2$

$$q = \frac{61.2}{45^2}\ \frac{4200}{204} = 0.62$$

$y\ k = 0.185$

de la Figura 10 (II-428) , $R = 0.30$

$$\Rightarrow M_R = R\, F_R\, b h^2 f''_c$$

$$= 0.3 \cdot 0.8 \cdot 45^3 \cdot 204$$

$$= 4461\ ton \cdot cm$$

$$\Rightarrow \frac{\Sigma M_{ecol}}{\Sigma M_{eviga}^{*}} = \frac{2 \times 4461}{1885 + \underset{(\phi M_n)}{0.9 \cdot 1127}} = 3$$ // { Para las crujías 14 y 18 la razón da 1.45 ✓

* se debería incluir $\alpha = 1.25$ p/vigas

b2) Traslapes

Sólo se permiten a la mitad central del elemento

$$\text{Traslape} \geq \begin{cases} 1.33\, L_d & \text{(según 3.9.1)} \\ (0.01\, f_y - 6)\, d_b \end{cases}$$

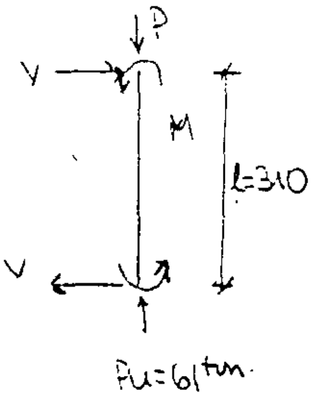
$$\text{Pero } L_d = L_{db} = 0.06\ \frac{a_s\, f_y}{\sqrt{f'_c}} \geq 0.006\, d_b\, f_y$$

$\Rightarrow$ Traslape $= 1.33 \cdot 0.06 \dfrac{5.1 \cdot 4200}{\sqrt{300}} = 99\text{ cm} \Rightarrow 1\text{ m}$

que debe ser mayor que $1.33 \cdot 0.006 \cdot 2.5 \times 4200 = 34$ ✓

b.3) Diseño por cortante

Las columnas se dimensionarán de manera que no fallen por fuerza cortante antes de que se formen articulaciones plásticas en las vigas (5.3.5)



Diseño por capacidad

$M = 0.5\,(1.5\,\Sigma M_{vigas})$

$M = 0.5\,[1.5\,(1885 + 0.9 \cdot 1127)]$

$= 2174$ ton·cm (que es $0.5\,M_{Rcol}$ aprox)

$V_u = \dfrac{2M}{l} = \dfrac{2 \times 2174}{310} = 14$ ton

Según 2.1.5

$V_{cR} = 0.5\,F_R\,bd\sqrt{f^*_c}\,(1 + 0.007\,P_u/A_g)$

para $\rho > 0.01$ ($\rho_{col} = 0.03$)

$= 0.5 \cdot 0.8 \cdot 45 \times 40 \sqrt{204}\,(1 + 0.007 \cdot 61000/45^2)$
$= 12.5$ ton

Pero $S = \dfrac{F_R A_v f_y d}{V_u - V_{cR}}$

$$\frac{A_v}{s} = \frac{V_u - V_{CR}}{F_R f_y d} = \frac{1500}{0.8\cdot 4200\cdot 46.5} = 0.0096 \frac{cm^2}{cm}$$

5.4) Por confinamiento (según 5.2.4)

$$\frac{A_{sh}}{s} = 0.3\left(\frac{A_g}{A_c}-1\right)\frac{f'_c}{f_y}h_c = 0.3\left(\frac{45^2}{39^2}-1\right)\frac{300}{4200}39 = 0.277\frac{cm^2}{cm}$$

$$\frac{A_{sh}}{s} = 0.12\frac{f'_c}{f_y}h_c = 0.12\frac{300}{4200}39 = 0.334\frac{cm^2}{cm}$$

$$h_c = 45 - \underset{2\times 3}{6} = 39\,cm$$

Rige la última, $\frac{A_{sh}}{s} = 0.334$

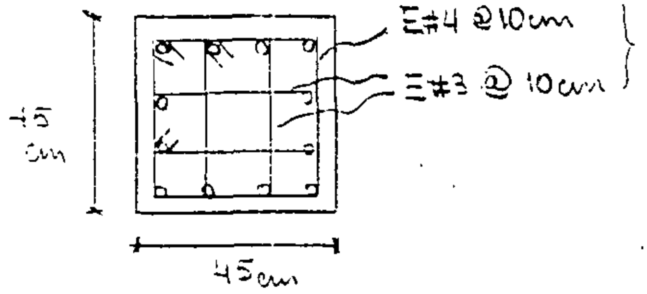
$s \leq$
- $h/4 = 11\,cm$ | según 5.3.4
- $10\,cm$

según 4.3.3
- $\frac{850}{\sqrt{f_y}} d_{b_{col}} = \frac{850}{\sqrt{4200}} \underset{\#8}{2.5} = 33\,cm$
- $48 d_{b_{estribo}} = 48\cdot 1.25 = 60\,cm$
- $h/2 = 23\,cm$

Rige $s = 10\,cm$

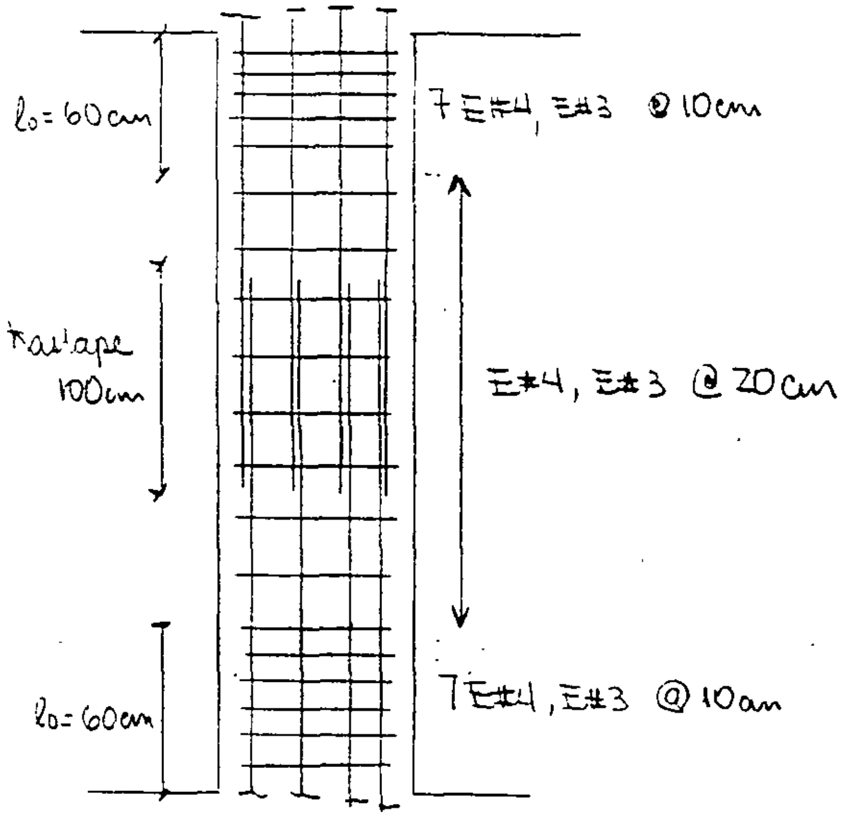
$\Rightarrow A_{sh} = 0.334\times 10 = 3.34\,cm^2$

Usar E#4, E#3 @ 10 cm $\Rightarrow A_{sh} = 4\,cm^2$



En una longitud

l_0 { 45cm
310/6 = 52 cm
60cm ✓

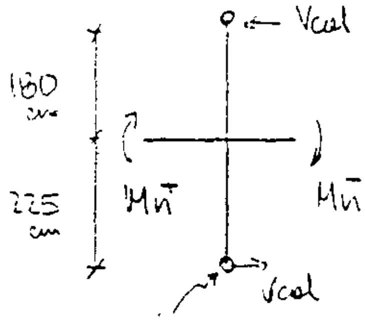


COLUMNA

1er piso

c) JUNTA VIGA-COLUMNA

c.1) Resistencia al corte



punto de inflexión a la mitad de la altura de la columna

Mn̄ $a = \frac{\overset{\alpha}{1.25}\, f_y A_s}{f''_c b} = \frac{1.25 \times 4200 \times 11.36}{204 \times 35} =$

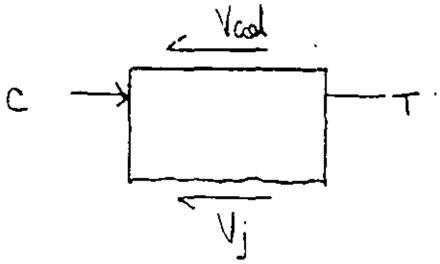
$a = 8.35$ cm

$Mn̄ = 1.25 \times 4200 \times 11.36 \left(46.5 - 8.35/2\right)$
$= 2524$ ton·cm

Mn̄ $a = \frac{1.25 \times 4200 \times 6}{204 \times 35} = 4.41$

$\Rightarrow Mn^{+} = 1395$ ton.cm

Pero $V_{col} = \frac{\Sigma Mn}{180+225} = \frac{2524+1395}{405} = 9.7$ ton



de M^{+}

$C = 1.25 \times 4200 \cdot 6.0 = 31.5^{ton}$
$T = 1.25 \times 4200 \cdot 11.36 = 59.6^{ton}$

$V_j = 31.5 + 59.6 - 9.7 = 81.4$ ton

Puesto que el nudo está confinado en 75% por el ancho de las trabes,

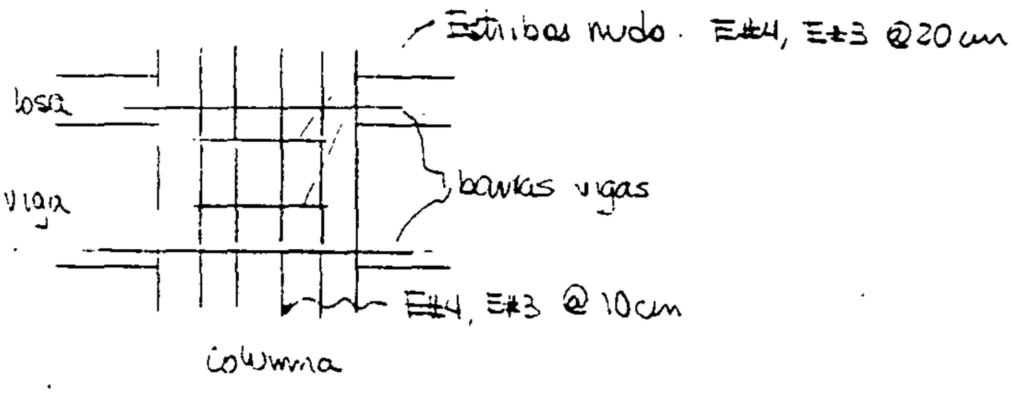
$V_{Rj} = 5.5 F_R \sqrt{f^{*}_c}\, b_e h$, $b_e = \frac{b_{viga} + b_{col}}{2} = \frac{45+35}{2} = 40$ cm

$= 5.5 \times 0.8 \cdot \sqrt{240} \times 40 \times 45$
$= 123$ ton $> V_j$ ✓

c.2) Refuerzo transversal de confinamiento

Puesto que el nudo está confinado por cuatro trabes, con relación de anchos viga/columna ≥ 0.75,

⇒ reducir el refuerzo transversal de la columna, con 2 juegos de estribos entre los lechos superior e inferior ← importante



c.3) Anclaje del refuerzo*

Según 5.4.4, puesto que los muros resisten más del 50% de la fuerza lateral,

$$\frac{h}{d_b} \geq 15$$

Para barras de columna

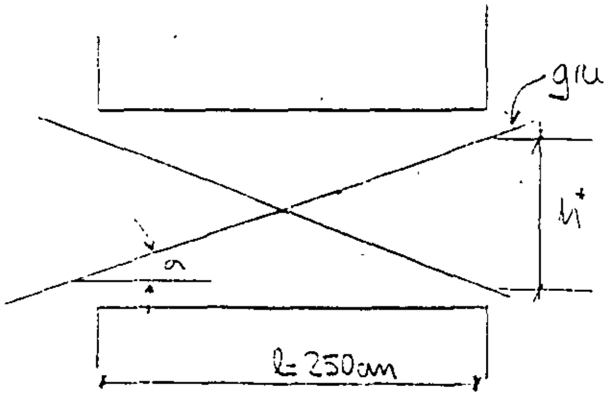
$$\frac{h_{viga}}{d_{b\,columna}} = \frac{50}{\underset{\hookrightarrow \#8}{2.5}} = 20 \checkmark$$

$$\frac{h_{columna}}{d_{b\,viga}} = \frac{45}{\underset{\hookrightarrow \#6}{1.9}} = 23 \checkmark$$

* a menudo controla el peralte, especialmente columnas

Según 5.4.1, se <u>deben</u> incluir dibujos acotados y a escala en los planos, del refuerzo en las uniones vigas-columna.

7) VIGAS DIAFRAGMA DE ACOPLAMIENTO

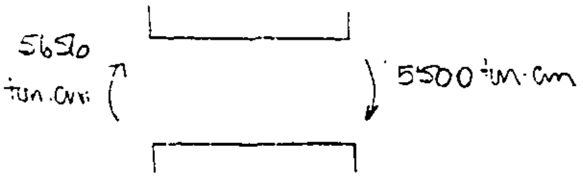


$M = T h^* \cos\alpha$

$V = (T+C)\,\text{sen}\,\alpha$

En nuestro caso. $l = 250$ cm, $h = 50$ cm $(l/h = 5 > 2)$

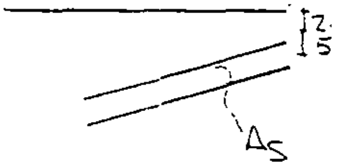
máximo para NTC DF (4.14.2)



Por paridad, $M = 5660$ ton.cm

$M_{máx} \approx \alpha M = 1.25 \times 5660 = 7075$

Si $h^* = 36$ cm y $\alpha = \text{ang tan } 0.2 = 11.3°$



$$T = \frac{M_{máx}}{h^* \cos\alpha} = \frac{7075}{36 \cdot \cos 11.3°}$$

$T = 200$ ton

$$\Rightarrow As = \frac{T}{f_y} = \frac{200}{4.2} = 48\ \text{cm}^2$$

usar 6#10

$As = 49.14\ \text{cm}^2$

(Rige sobre $V = (T+C)\,\text{sen}\,\alpha$, $T = 144$ ton)

Las diagonales se anclarán según 3.1.1c) multiplicado por 1.5

$$1.5L_d = 1.5 \cdot 0.06 \frac{a_s f_y}{\sqrt{f'_c}} \geq 1.5 \cdot 0.006\, d_b f_y$$

$$1.5 \cdot 0.006 \cdot 3.175 \cdot 4200 = 120cm$$

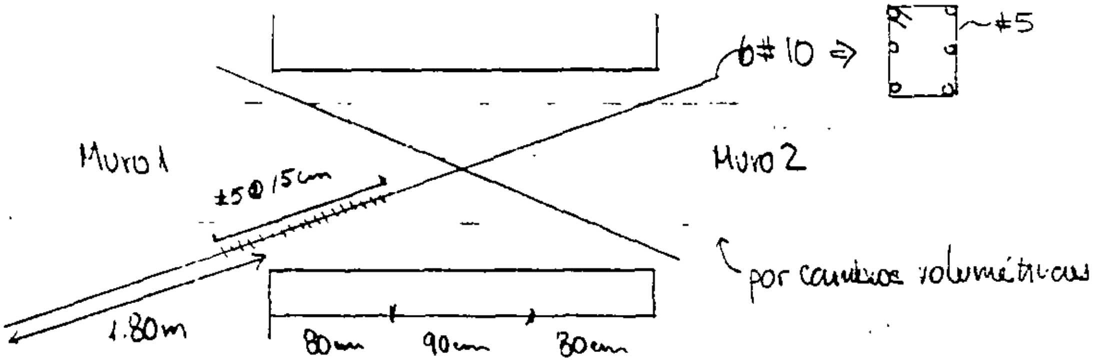
$$1.5 \cdot 0.06 \frac{8.19 \cdot 4200}{\sqrt{300}} = 178cm \checkmark$$

Estribos:

$s \leq$

- $\frac{850}{\sqrt{f_y}} d_b = \frac{850}{\sqrt{4200}} 3.175 = 42cm$
- $48 d_{b_{estribo}} = 48 \cdot \frac{5}{8} \cdot 2.54 = 76.2cm$
- $35/2$: 17.5 cm rige, usar s= 15cm

↑ menor dimensión



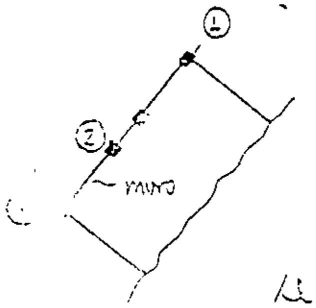
El resto de la viga lleva refuerzo por cambios volumétricos.

si $\rho = 0.002$ (3.10), $A_s = 50 \times 35 \times 0.002 = 3.5 cm^2$

⇒ usar #2.5 @ 50cm (verticales) no dibujados
2#3 horizontales

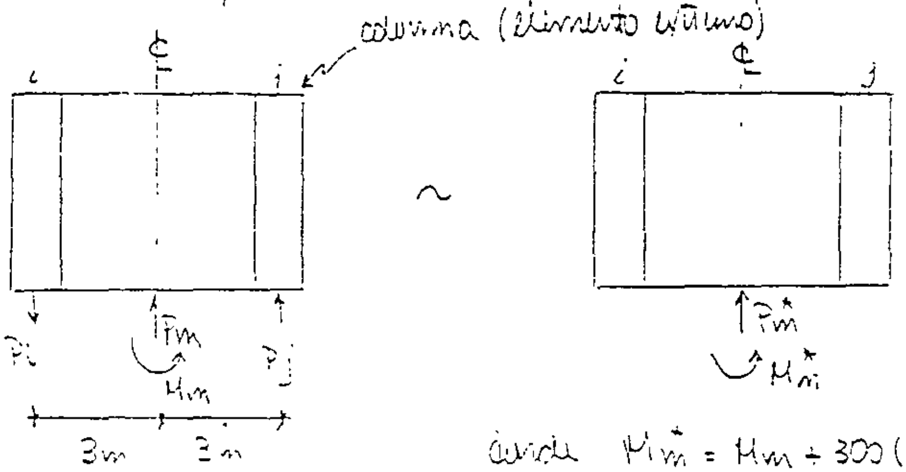
e) MURO DE CONCRETO

Se diseñará el muro de la planta baja entre columnas 1 y 2.



Del dimensionamiento: $t = 20$ cm
$h = 45$ cm

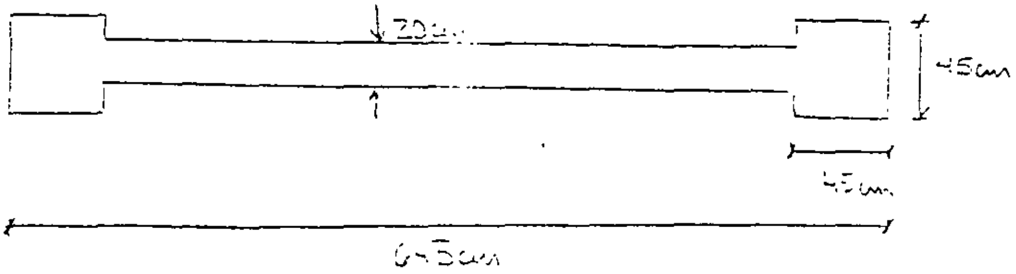
De acuerdo al modelo de la estructura, se obtuvieron elementos mecánicos en las columnas y en el muro. Se debe incorporar la carga axial de las columnas en la carga axial y flexión del muro. De esta forma.



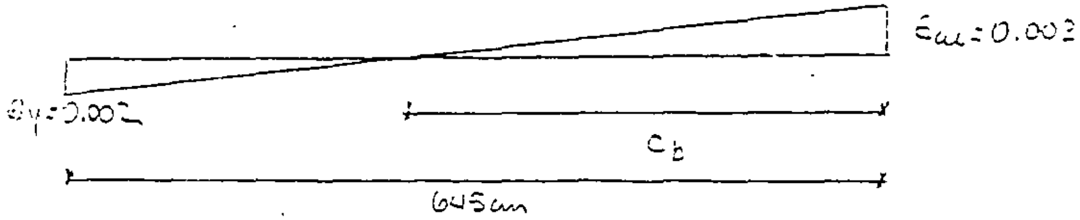
donde $M_m^* = M_m + 300(P_i + P_j)$
$P_m^* = P_m - P_i + P_j$

La carga de compresión más alta fue de 362 ton y de tensión de 143 ton.

Para encontrar la peor condición de carga, consideramos carga axial. Estimaremos en forma aproximada, la carga axial balanceada



(3)



$$\frac{0.003}{C_b} = \frac{0.002}{645 - C_b}$$

$$\Rightarrow C_b = \frac{0.003(645)}{0.005} = 387\text{ cm}$$

La profundidad del bloque equivalente de esfuerzos es de

$$a_b = 0.8 \cdot C_b = 309.6\text{ cm}$$

La carga axial balanceada será.

$$P_b = \underbrace{0.85\, f'_c}_{f''_c}\, a\, b + A'_s f_y - A_s f_s$$

Suponiendo que $A_s' = A_s$, $f_y = f_s$ y $b = 20$ cm.

$$P_b = 204 \times 309.6 \cdot 20 \,/\, 1000$$

$$= 1263^{\text{ton}} \text{ (compresión)}$$

** Es claro que la máxima carga de compresión está muy por debajo de P_b

$\Rightarrow$ la condición crítica de carga es

$$M_u = 302{,}350 \text{ ton cm}$$

$$P_u = 143^{\text{ton}} \text{ (tensión)}$$

V_u: 29.4^{ton} (se verificará por capacidad) e.1) Diseño por cortante

La fuerza cortante $V_u = 294^{ton}$ es constante en la altura del muro en el primer entrepiso

Verifiquemos que $V_{max} > V_u$ (4.5.2 NTC-Concreto)

$$V_{u\,max} = 2 F_R L t \sqrt{f^*_c} = 2 \cdot 0.8 \cdot 645 \cdot 20 \sqrt{240}/1000$$

$$= 320^{ton} > 294 \text{ ton} ✓$$

Puesto que $\frac{H}{L} = \frac{2250}{645} = 3.5 > 2$, el cortante que toma el concreto se evalúa con la ec. 2.18 (para vigas), afectada por un factor que considere la carga axial a tensión

$$V_{CR} = 0.5 F_R \, bd \sqrt{f^*_c} \; \underbrace{(1 - 0.03 \, {}^{P_u}/_{A_g})}_{\text{flexotensión}} \quad \text{si } \rho \geq 0.01$$

donde $b = 20$ cm

$d = 0.8L = 0.8 \cdot 645 = 516$ cm

$$\frac{P_u}{A_g} = \frac{143000}{2 \cdot 45^2 + 20(600-45)} = 9.4 \text{ kg/cm}^2$$

$$V_{CR} = 0.5 \cdot 0.8 \cdot 20 \cdot 516 \cdot \sqrt{240} \; (1 - 0.03 \cdot 9.4)/1000$$

$$V_{CR} = 45.8 \text{ ton}$$

El acero horizontal se determina como (ec. 4.6)

$$\rho_h = \frac{V_u - V_{CR}}{F_R f_y d t} = \frac{294000 - 45500}{0.8 \cdot 4200 \cdot 516 \cdot 20} = 0.0072 > \rho_{min} ✓$$

donde $\rho_{min} = 0.0025$

Nota. V_{cR} se determinó con 2.18 para $\rho \geq 0.01$, pero $\rho_h < 0.01$

usando 2.17 con $\rho_h = 0.007$

$$V_{cR} = 38.2^{\text{ton}}$$

$$y \quad \rho_h = \underline{\underline{0.0074}} \quad .$$ Poco sensible.

Si $t = 20$ cm y $\rho_h = \dfrac{A_{vh}}{S_h t}$

$$\frac{A_{vh}}{s} = 0.148 \ \frac{\text{cm}^2}{\text{cm}}$$

Puesto que $t > 15$ cm, se colocará el refuerzo en 2 capas. Si se usan 2#5 ($A_{vh} = 4$ cm^2),

$$S = 4/0.148 = 27 \text{ cm}$$

$$S = 25 \text{ cm} < s_{max} = 25 \text{ cm} \checkmark$$

$\therefore$ usar $\underline{2\#5 \ @ \ 25 \text{ cm}}$

El acero vertical se calcula con

$$\rho_v = 0.0025 + 0.5\left(2.5 - \underset{\substack{\uparrow \\ \text{notar}}}{\frac{H}{L}}\right)\left(\rho_h - 0.0025\right)$$

$\rho_v = 0.0025 - 0.5 (2.5 - 2.5) (0.0074 - 0.0025)$

$= 0.0001 < \rho_{min}$

usar $\rho_v = \rho_{min} = 0.0025$

Similarmente al desarrollo de ρ_h

$\frac{A_{vv}}{s} = 0.05$

Si $s_{max} = 35 cm$, $A_{vv} = 0.05 \times 35 = 1.75 cm^2$

$\Rightarrow$ usar 2#4 @35 cm
($A_w = 2.53 cm^2$)

e.2) Revisión del elemento extremo como columna corta

Sin considerar el acero del alma, se diseñará el elemento extremo para la máxima carga axial. De las varias combinaciones de carga, considerando carga axial y momento, P= 537 ton
$\hookrightarrow M = P \cdot L$

$A_g = 45^2 = 2025 cm^2$

Si usamos 12 #10, $A_{st} = 12 \cdot 8.19 = 98.30 cm^2$

($\rho_t = 0.048 < 0.06$)

$P_{u0} = 0.8 [f'_c (A_g - A_{st}) + f_y A_{st}]$

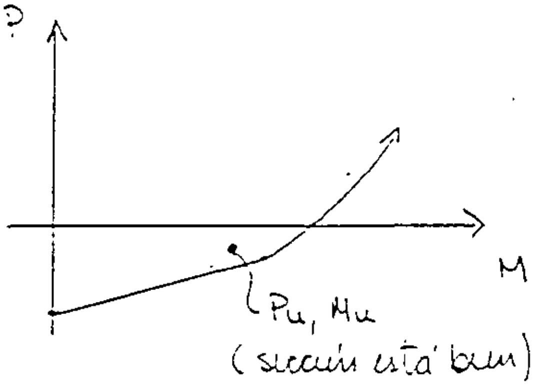
$= 0.8 [204 (2025 - 98.3 - 4200 \cdot 98.3] /1000$

$= 645^{ton} > P_u$

e.3) Revisión de la capacidad

Si calculamos algunos puntos del diagrama de interac-ción:

	P [ton]	M [ton·cm]	
[illegible]	145	342782	← Mn
	0	385257	
	136	424600	
	318	475670	
	454	512901	
	590	548846	
	726	583444	

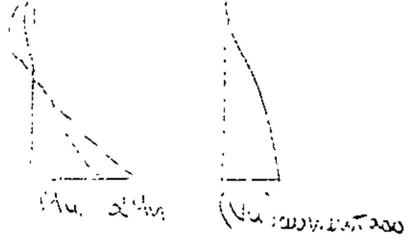


Según la condición de carga crítica
$V_u = 294$ ton

$$(V_u)_{aumentado} = V_u \frac{\alpha M_n}{M_u} \quad ; \quad \alpha = 1.1 \text{ a } 1.2$$

$$\frac{1}{Fe} = \frac{1}{0.9}$$

$$= 294 \frac{1.1 \cdot 342752}{302550}$$



$$= 1.25 V_u = 367 \text{ ton}$$

Pero la capacidad

$$\phi V_n = V_{cr} + V_s = 45.8 + \rho_h f_y d t \qquad \text{donde } \rho_h = \frac{A_v}{S t} = \frac{4}{25 \cdot 20} = 0.008$$

$C_{1.0}$

$$= 45.8 + 0.008 \cdot 4200 \cdot 516 \cdot 20 / 100$$

$$= 45.8 + 347$$

$$= 393 \text{ ton} > (V_u)_{aumentado} \ //$$

e.4) Confinamiento de los elementos extremos

Similar a las columnas, sólo que el refuerzo horizontal se coloca a lo largo del elemento con la misma separación.

$\frac{A_{sh}}{s} = 0.334$; si rige $s = 10$ cm.

$A_{sh} = 3.34$ cm^2

$\Rightarrow$ 2#4, 2#3 @ 10 cm

e.5) Uniones de barras (traslapes) L_t

- a la mitad de la altura, y barras alternadas

Para las barras del #10, $L_t = 1.33 L_{dc}$

donde $L_{dc} = \frac{0.06 \cdot a_s f_y}{\sqrt{f'_c}} = \frac{0.06 \cdot 5.19 \cdot 4200}{\sqrt{300}} = 119$ cm

que tiene que ser mayor o igual que $0.006 d_b f_y = 0.006 \cdot \frac{10}{8} \cdot 2.5 \cdot 4200$
$= 75.8$ cm

$\Rightarrow L_t = 1.33 \cdot 119 = 160$ cm

Para las barras del #4, $L_t = 40$ cm | máx $(0.006 d_b f_y)$

Para el anclaje de las barras del # 5 (horizontales)

Según 3.1.1

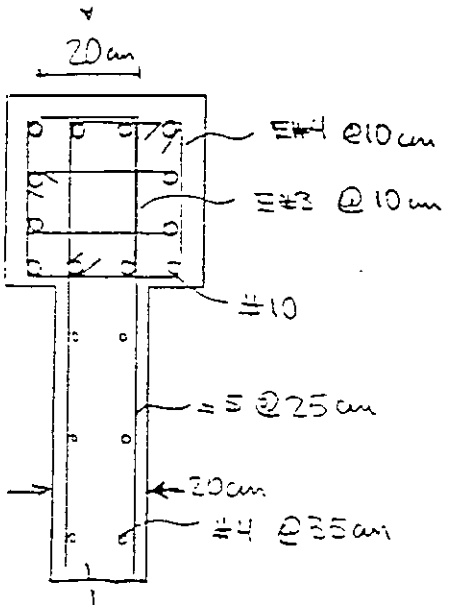
$$L_{dh} = \frac{0.076\, d_b f_y}{\sqrt{f'_c}} = \frac{0.076 \times \frac{5}{8}\, 2.54 \times 4200}{\sqrt{300}}$$

$= 30\text{cm} < 45\text{cm}$ (ancho del elemento de extremo)

El tramo recto después del doblez será de $12 d_b$ = 19cm

usar 20cm

Croquis

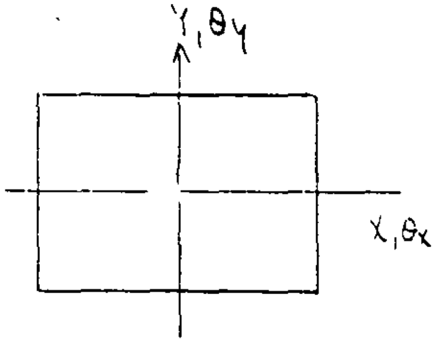


Elemento extremo
45cm × 45cm

6.7. INTERACCIÓN SUELO-ESTRUCTURA

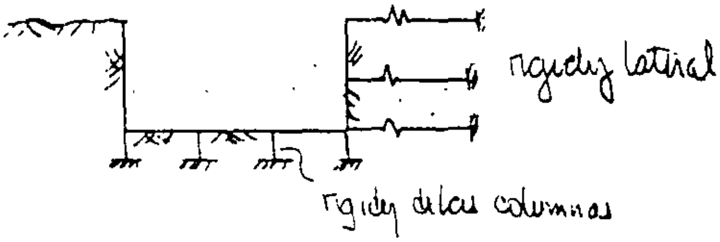
- losa de cimentación y muros de contención, que se suponen rígidas
    - pilotes
- contratrabes

Algunos comentarios sobre modelación de la interacción suelo-estructura usando ETABS.



El objetivo es modelar k_x, $k_{\theta x}$, k_y y $k_{\theta y}$

Para cajones de cimentación.



La rigidez lateral total de la cimentación está dada por:

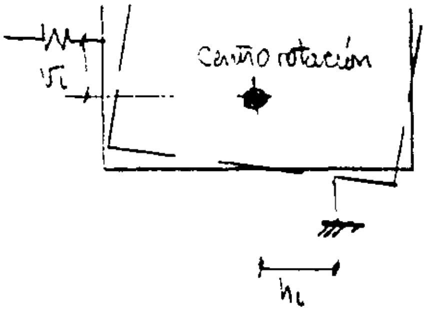
$$K_x = \sum k_{lateral} + \sum \frac{12EI}{L^3(1+2\beta)}$$

columnas equivalentes

donde $\beta = \frac{6EI}{L^2 A_r G}$ (si $\beta = 0$: deformaciones por cortante son nulas)

La rigidez rotacional total se obtiene de:

$$k_{\theta\theta} = \sum k_{lat} v_i^2 + \sum \frac{AE}{L} h_i^2$$ (contribución de la rigidez axial de los resortes-columna)



$$+ \sum \frac{12EI}{L^3(1+2\beta)} v_i^2$$ (contribución de la rigidez lateral de las columnas)

$$+ \sum \frac{2EI(2+\beta)}{L(1+2\beta)}$$ (contribución de la rigidez rotacional de las columnas

El término $k_{x\theta}$ se obtiene

$$k_{x\theta} = \sum k_{lateral} v_i + \sum \frac{12EI}{L^3(1+2\beta)} v_i + \sum \frac{6EI}{L^2(1+2\beta)} v_i$$

Se puede suponer que el centro de rotación estará a 2/3 de la profundidad de la cimentación

Se puede incrementar la rigidez lateral en forma lineal con la profundidad de la cimentación

$$k_{lateral} = k_{o\,lateral} + c\,y$$

Esquemáticamente

K_x, K_y, $K_{\theta x}$, $K_{\theta y}$ se obtienen: RDF-87, ATC, etc.

Para obtener klat. y k columna-resorte, se supone $\beta=0$ y $L= 0.5$ cm (no afecta los elementos mecánicos en la superestructura)

Se resuelve un sistema de ecuaciones de la forma

$$\left\{\begin{matrix} k_{0x} \\ k_x \\ k_{0y} \\ k_y \\ k_{0x} \end{matrix}\right\} = \left[\begin{matrix} & & \\ & * & \\ & & \end{matrix}\right] \left\{\begin{matrix} k_{0x} \\ A \\ I_y \\ k_{0y} \\ I_x \end{matrix}\right\}$$

* ver ecuaciones pasadas, para una posición de CRotación

$$\downarrow$$

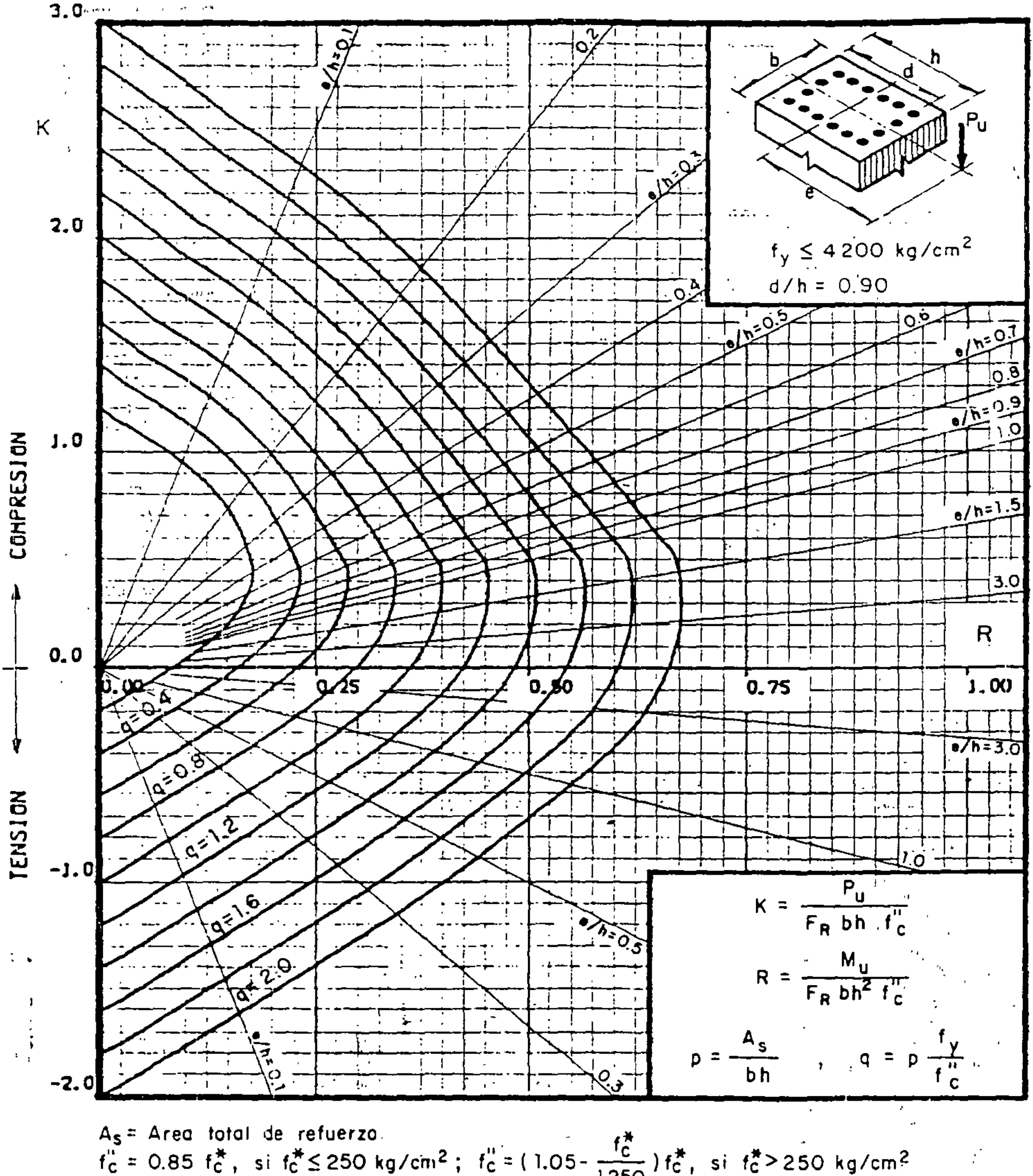
estos valores se usan en ETABS.

Nota: un cambio en CR, produce cambios de signo en el determinante de la matriz *, además, si un modo de vibrar rota con respecto a un punto para el cual la matriz es singular, se producen problemas numéricos (inestabilidad).

Aún si $\det|*| \gg 0$, si se usan valores de k_{0x}, A, ..., muy grandes se pueden tener problemas numéricos

Puesto que la losa de cimentación es flexible, se produce una reducción de la rigidez de desplazamiento lateral de las columnas que puede llegar a causar problemas numéricos

Para este edificio, el periodo fundamental aumentó en 20% en relación al calculado suponiendo base rígida



A_s = Area total de refuerzo

$f''_c = 0.85\, f^*_c$, si $f^*_c \le 250$ kg/cm²; $f''_c = (1.05 - \frac{f^*_c}{1250})\, f^*_c$, si $f^*_c > 250$ kg/cm²

F_R = Factor de reducción de resistencia

P_u = Carga axial última

M_{uy} = Momento flexionante último en dirección y = $P_u \cdot e_y$

M_{ux} = Momento flexionante último en dirección x = $P_u \cdot e_x$

Fig 10

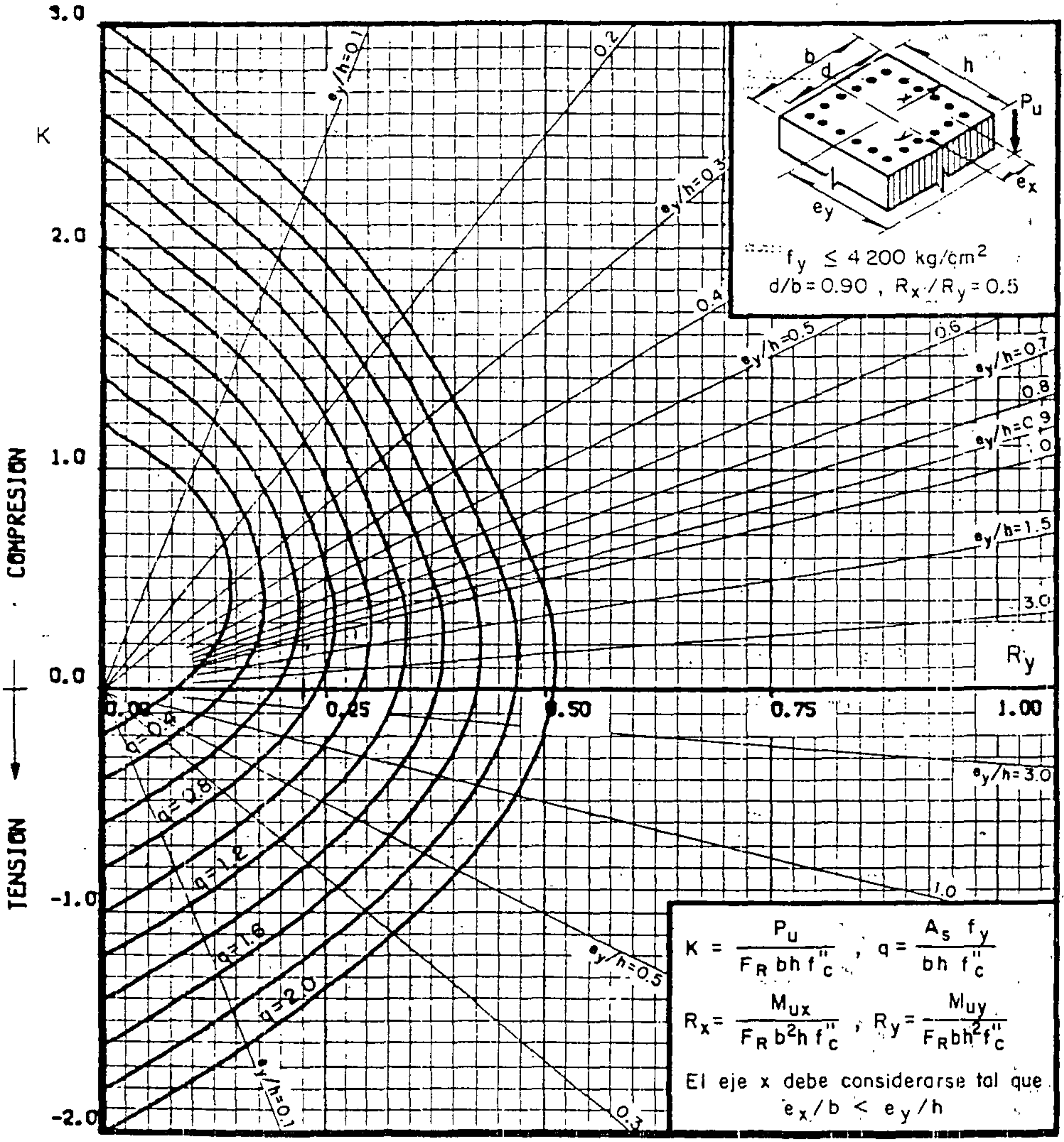


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Fig 42